WATERPOWER

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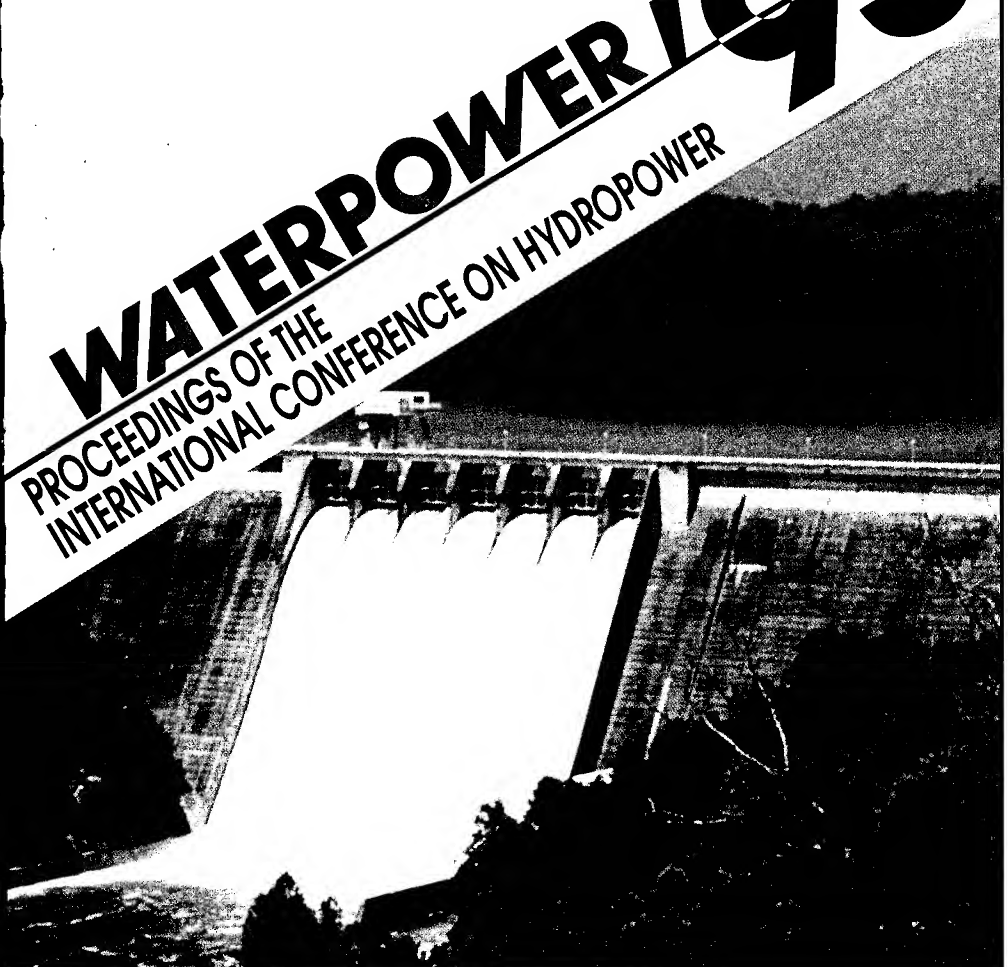
WATERPOWER '93

PROCEEDINGS OF THE INTERNATIONAL CONFERENCE ON HYDROPOWER



VOLUME 1

Edited by W. David Hall

WATERPOWER '93

PROCEEDINGS OF THE INTERNATIONAL CONFERENCE ON HYDROPOWER

Nashville, Tennessee
August 10-13, 1993

Edited by W. David Hall

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Volume I

Economics & Finance
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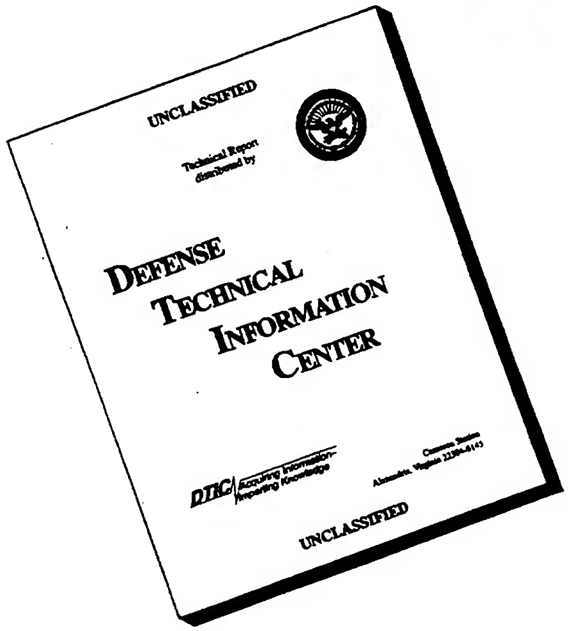

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ABSTRACT

This proceedings contains the papers presented at the Waterpower '93 conference held in Nashville, Tennessee, August 10-13, 1993. The conference brought together owners, planners, engineers, regulators, manufacturers, and others to share vital information surrounding the conference theme, Hydropower - Its Role in World Energy. This eighth edition of the conference series which began in 1979 was hosted by the Tennessee Valley Authority and the U.S. Army Corps of Engineers, Nashville District. Subject areas include: 1) environmental issues; 2) legal factors; 3) planning; 4) hydraulics; 5) hydrology; 6) operation and maintenance; 7) rehabilitation; 8) research and development; 9) computer applications; 10) geotechnical, mechanical, and electrical systems; 11) reservoir system operation; and case works.

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PREFACE

Since the first Waterpower Conference in 1979, Waterpower has convened every two years. This conference has become the definitive forum for sharing of information and discussing issues concerning hydroelectric power. This eighth edition of the biennial gathering was held in Nashville, Tennessee on August 10–13, 1993.

The perceived simplicity of production of hydropower gives it a less dramatic public image than that of some other sources of energy. While hydropower is the most efficient energy technology, opportunities exist to improve its performance and its existence with the environment. Water turned the wheel which powered the mill long before electric generators were envisioned, and the extension of water power to provide electricity is often viewed as simplistic. David E. Lilienthal, one of the Tennessee Valley Authority's first Directors, fifty years ago called hydropower "an outdated source of energy whose time has not yet come." Perhaps hydropower's time is here!

Each of the papers included in the proceedings has received a positive peer review and has been accepted for publication by the proceedings editor. All papers are eligible for discussion in appropriate journals, and all papers are eligible for ASCE awards.

Waterpower '93 continued two innovations initiated at the Denver conference of 1991. The poster forum again showcased graphical or pictorial depiction of a number of projects. Poster presenters were invited to prepare papers to accompany their posters, and those papers are published along with all other conference papers in this proceedings.

Again the Technical Program Committee presented a Best of Waterpower award to one of the paper authors. This award recognizes a paper which uniquely advances the interests of hydropower as demonstrated by the timeliness of the subject matter, advancement in technology, innovation, and measurable benefits.

W. David Hall, Chair
Technical Program Committee
WATERPOWER '93

ACKNOWLEDGEMENTS

The success of WATERPOWER '93 Conference is attributable to the dedicated people working on the Steering Committee and the Technical Program Committee. The Steering Committee provides policy guidance and oversight for the planning of the Conference. The Technical Program Committee sets the agenda to achieve the goal of soliciting, selecting, and presenting suitable papers for the conference attenders. The organizations assisting in the conference, and the members of each committee are:

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INTRODUCTION

WATERPOWER '93 is organized around 54 sessions with four papers typically being presented in each session and a paper poster forum. The Technical Program Committee selected the papers to be presented at the Conference from the response of candidates to the WATERPOWER "Call for Papers" circulated in May 1992. Topical areas are identified to categorize each paper so that like papers share the same session. The papers in the three volume proceedings follow this same format. That is, the papers are grouped by topic, sub-topic, and session number. The topics in each volume are listed on the spine and the title page. There are nineteen topic areas with volume 1 containing 5 topics, volume 2 containing 9 topics, and volume 3 containing 5 topics and the papers presented in the poster forum. The complete table of contents and indices appear in each volume. There are two indices, one covering subject matter and the other listing authors.

WATERPOWER '93 Nashville, Tennessee

TECHNICAL PROGRAM – OVERVIEW

Date	Day	Time	Carrol Room	Browning A Room	Browning B Room	Taylor B Room	Taylor A Room	Handy Room
August 10		5:30 - 8:30PM	Welcome Reception					
August 11	WEDNESDAY	8:30 - 10:00AM	OPENING PLENARY					
	WEDNESDAY	10:00 - 10:45	*break*					
	WEDNESDAY	10:45 - 12:15	ENVIRONMENTAL-1 Fish Passage Session 1	DAM SAFETY-1 Analysis Session 2	REHABILITATION & MODERNIZATION-1 General-1 Session 3	OPERATION & MAINTENANCE-1 General Maintenance-1 Session 4	CASE STUDIES-1 Project Evaluation and Assessments Session 5	HYDRAULICS-1 Design of Hydraulic Structures Session 6
	WEDNESDAY		*lunch*					
	WEDNESDAY	1:45 - 3:15	ENVIRONMENTAL-2 Turbine Impacts on Fish Session 7	DAM SAFETY-2 Dam Rehabilitation Session 8	REHABILITATION & MODERNIZATION-2 General-2 Session 9	COMPUTER APPLICATIONS-1 (Pumped Storage & Project Systems) Session 10	CASE STUDIES-2 Innovations in Equipment and Design Session 11	HYDRAULICS-2 Modeling & Analysis Session 12
	WEDNESDAY	*visit poster forum*	*break*					
	WEDNESDAY	4:00 - 5:30	ENVIRONMENTAL-3 Fish Protection Session 13	LICENSING/ LEGAL-1 Relicensing Issues Session 14	RESEARCH & DEVELOPMENT-1 Hydraulic Turbine Performance Session 15	OPERATION & MAINTENANCE-2 General Operations Session 16	CASE STUDIES-3 General-1 Session 17	TURBINES-1 Design Session 18
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	THURSDAY		*lunch*					
	THURSDAY	1:45 - 3:15	ENVIRONMENTAL-6 (EPRI) Diversion Screens Session 31	LICENSING/ LEGAL-2 Licensing Issues Session 32	HYDROLOGY-1 Streams and Reservoirs Session 33	ELECTRICAL SYSTEMS & CONTROLS Session 34	CIVIL WORKS-1 General-1 Session 35	TURBINES-3 Testing Session 36
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Hydro - "Rest-of-System" Integration Modeling
for
Analyzing Relicensing and Redevelopment Options

Mark I. Henwood[1], David K. Hoffman[2], Roger L. Raeburn P.E.[3]
February 1, 1993

Abstract

Modernization, redevelopment, and relicensing of a complicated hydroelectric project such as PacifiCorp's North Umpqua project, containing cascaded generating stations and storage reservoirs of varying size, can require the study of numerous options. The authors have recently developed the WaterWay module of the Multisym chronological production simulation model to perform the type of detailed analyses required to analyze options associated with these processes. This module is a completely generalized routing model tailored to hydroelectric operations and it is fully capable of representing complex systems. The resulting modeling system permits evaluation of hydroelectric facilities in the context of overall electric system operation. This paper describes this modeling system, the types of studies performed, and presents hourly results from a ramp rate constraint study.

Introduction

PacifiCorp has recently initiated a program involving a detailed evaluation of the North Umpqua Project facilities to assess current capabilities and potential modifications that may be considered for improvement of project operations. The goal of this program is to identify physical and operational modifications which will allow for the improved utilization of the existing hydrologic resource and shifting power generation into on-peak periods, while taking into consideration changing environmental considerations and requirements.

Three principal mechanisms are being considered to improve utilization of the project's hydrologic resource: increasing the quantity of water utilized by the project; raising the efficiency of this water utilization; and, shifting the

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1 President, Henwood Energy Services, Inc, Sacramento, CA
2 Partner, The Simulation Group, Sacramento, CA
3 Manager, Hydro Engineering, PacifiCorp, Portland, OR

project power generation into daily or weekly on-eak power periods. Alternatives being considered to increase the water utilized by the project include potential diversion of new stream segments into the project conveyance system. The shifting of project power generation into the on-peak periods can be accomplished with an increase in storage facilities allowing them to handle additional inflows and provide greater regulation of flows. Rehabilitation and upgrading of generating equipment throughout the project is being evaluated to increase unit efficiency.

Changes in project operations are also being evaluated for the current project and in conjunction with the alternative modifications being considered within the context of these studies. The evaluation of the overall project operation is an extremely important part of the program, as any physical project modifications considered must be evaluated from an operational compatibility standpoint as well as the functional assessment.

The large number of alternatives involved (approximately 43) and the inner-dependent relationship of many of them pose an analytical challenge. The problem of assessing the benefits of individual alternatives in the context of system operations is compounded when the interaction of multiple changes are considered. To handle these problems it became apparent that a simulation model that could both model the current system and also be capable of optimizing changes to the project was needed.

To meet these challenges, the Multisym[4] chronological production simulation model is being enhanced via the WaterWay module to perform these types of detailed analyses. The general modeling approach used is to decompose the electric system analysis problem into hydro and "rest-of-system" sub problems. The "rest-of-system" sub-problem is actually the simulation of the entire electrical production system, producing either a (1) cost grid or (2) system load as the optimization target for use in hydro scheduling. The cost grid accounts for transmission restrictions, other hydro resources, sales, and purchases as well as thermal unit operations. In the event a system load is used as the optimization target, time varying resources may be used to modify the loads presented to the hydro system.

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4 Multisym is a detailed, multi-area, chronological production simulation model used by utilities, consultants, and agencies for the analysis and planning of electric utility operations. The model is developed and supported by The Simulation Group, Sacramento, California.

The general coordination between the "rest of system" and the WaterWay module is shown in Figure 1.
The authors are supporting the use of this integrated system by PacifiCorp for modernization, redevelopment, and relicensing planning and analysis. The new system is already allowing for realistic evaluation of redevelopment options in the context of the extremely complex system operations in the Pacific Northwest. The balance of this paper discusses the system modeling problem, the algorithms used in the WaterWay model for performing both simulation studies or optimization studies, and the use of the model for analyzing a complex hydro system.

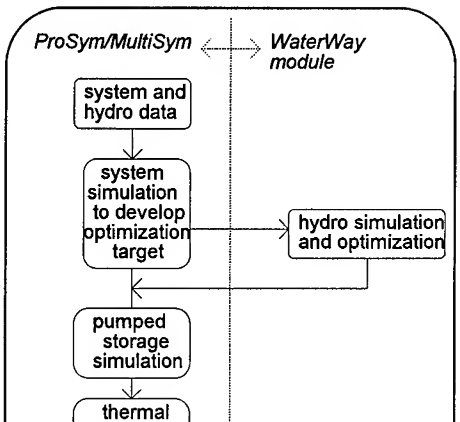

Figure 1
Block Diagram

Power System Modeling

Simulation models are frequently used in the electric utility industry for comparison of alternatives. The simulation models generally fall into two general classifications: load duration models and chronological models. Load duration curves models employ a mathematical procedure which provides for an efficient treatment of uncertainty in generation unit availability but at the cost losing the ability to precisely represent chronological constraints such as minimum up times, minimum down times, ramp rates, and startup costs. Chronological models simulate system operation on an hourly basis and use Monte Carlo techniques to represent uncertainty.

A common resource planning procedure is to compare the total electric system operating and capital costs resulting from the various alternatives with a "base" case. Simulation models estimate the system operating cost by solving a non-linear minimization problem defined as:

Minimize:

$$\text{system cost} = \Sigma_t \Sigma_i \; [\, f_{it} + vom_{it} + ext_{it} + sc_{it} \,]$$

Where:

- i station index for all stations, i.e. hydro, thermal, purchases, renewables, pumped storage, etc.
- t hour index
- f_{it} fuel or purchase power cost of station i in hour t
- vom_{it} variable operation and maintenance cost of station i in hour t
- ext_{it} emission cost of station i in hour t
- sc_{it} startup cost of station i in hour t

Subject to:

1. power balance constraints
2. operating reserve constraints
3. maximum and minimum generation constraints
4. minimum up/down time constraints
5. ramp rate constraints
6. limits on available energy

Hydroelectric resources are usually the lowest cost resource for a system but are subject to limitations on energy availability. A frequently used heuristic for approximating the optimum use of energy limited, peaking hydro plants is to utilize a load leveling algorithm. The goal of this approach is to minimize the highest load during the optimization period (frequently a week). Figure 2 shows an example load curve which has been "levelized" by hydro generation.

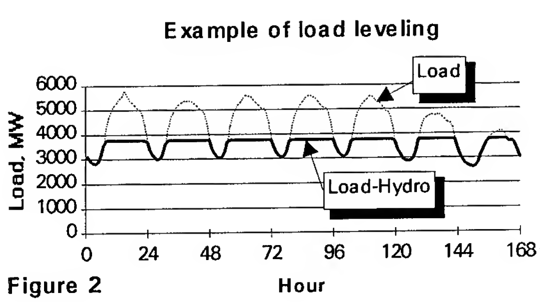

Figure 2

Load leveling (or peak shaving) is a useful objective function because due to the economic dispatch of the thermal system, periods with higher loads typically also have a higher incremental cost. However, as a result of time varying resource availability and chronological constraints on system

operations, system costs may not be monotonicly increasing with system load. For this situation, the design provides for the use of the Multisym model to calculate an alternative optimization target, a marginal cost array which specifies the incremental value of reducing the "rest of-system" dispatch in any hour. Because of these considerations WaterWay is designed for use of either a load curve or the cost array as an optimization target when run in optimized mode.

WaterWay

The WaterWay module is a completely generalized routing model tailored to complex hydroelectric operations. The WaterWay design specifies execution in three separate modes:

- a standalone mode, where operation of the hydro system is accomplished via a set of control functions which implement peaking operations, set release operations, and combinations of the above;

- a optimized standalone mode where the operation of the hydro system is scheduled by a network flow programming algorithm [1] in conjunction with an intelligent search used to address the unit commitment problem. In this mode, hydro units may either be driven by control functions, or optimized; and

- an integrated operating mode where the hydro-"rest of system" coordination problem is progressively solved through time. Again, hydro units may either be driven by control functions, or optimized.

The model objective is to simulate the hourly operation of a network of hydroelectric generation stations over a 168-hour time period given (1) a set pattern of storage releases, (2) a control function, or (3) use of the optimization routine for operation of each storage release, and initial conditions on storage and previous flows. Initial conditions are either input by the user or carried over from the previous week or year of a multi period simulation.

The simulation model is constructed as a linked set of "standard" hydrologic units. These standard units, as shown in Figure 3, consist of:

1. a summing junction
2. a storage reservoir with evaporation losses and seepage
3. river channel releases and spill
4. controlled diversions with optional generation
5. natural inflow and other water demands
6. flexible linking of diverted and river flows, through time delays, to arbitrary summing junctions
7. generation units with headwater, tail water, head loss, and efficiency curves
8. the ability to link a standard unit to an uphill unit creating a pumpback unit

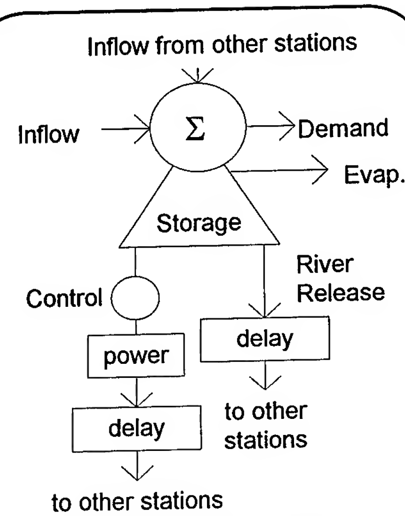

Figure 3 - Standard Hydro Unit

The standard units are subject to the following general constraints:

1. conservation of water at each summing junction/storage reservoir
2. bounds on storage, flows, and power generation
3. observation of release priorities in standard order
4. ramp rate constraints on diversion stated as either absolute flow change limits, percentage flow change limits, or limits on the change permitted in river elevations.

The general solution used in the optimization mode[5] is:

1. decomposition of the hydro-"rest of system" problem with the solution of the sub problems coordinated with either load information or incremental cost information
2. mapping of the hydro sub-system into a network flow model
3. solution of the hydro dispatch via an extended convex cost network flow programming algorithm
4. optimization of the hydro unit commitment via an intelligent search
5. final solution adjustment for constraint releases

---

5 a similar approach is used in [2].

The standalone simulation mode of WaterWay has been developed, tested, and is being used for analysis of the North Umpqua system located in the Pacific Northwest. The optimized mode of WaterWay is under development as of the writing of this paper.

The North Umpqua

The North Umpqua Project consists of eight hydroelectric power developments on the North Umpqua River watershed in Douglas County, Oregon. The project includes of a series of dams and canals on the North Umpqua River and its tributaries which divert water to the eight power developments. In total, the project contains 28.9 miles of canals, 8.6 miles of flumes and 5.3 miles of penstocks and tunnels for a total waterway length of 43.9 miles. Two major manmade reservoirs, Lemolo Lake and Toketee Lake, provide water storage for project operation. Total rated capacity of the project is 185,500 kW. The project was constructed between 1948 and 1955 and is owned and operated by PacifiCorp, d.b.a. Pacific Power & Light Company. Table 1 provides summary statistics regarding the project.

Elevations throughout the project area range from 1,600 feet to 9,182 feet. The entire basin is extremely rugged, characterized by precipitous slopes and steep stream gradients. Much of the entire drainage area is blanketed with a surface layer of granular pumice up to several feet thick. This material absorbs a relatively high proportion of rainfall and snow melt, resulting in runoff hydrographs of long duration and relatively low peaks providing the North Umpqua River basin with relatively uniform seasonal flow.

Table 1
North Umpqua Project Summary

power station/ storage facility	generation diversion (cfs)	head (ft)	storage (ac-ft)	rating (kW)
Lemolo Lake	n/a	n/a	12,350	n/a
Lemolo 1	565	710	0	29,000
Lemolo 2	655	705	237	33,000
Toketee Lake	1,250	420	1420	42,500
Clearwater 1	350	627	154	15,000
Clearwater 2	485	742	58	26,000
Fish Creek	150	995	83	11,000
Slide Creek	1,500	166	0	18,000
Soda Springs	1,600	107	660	11,000

Figure 4 presents a simplified representation of the relation of the major storage facilities and the power generation facilities. The WaterWay representation of this system contains 22 standard stations and 17 inflow files. This representation captures all storage facilities, timing of water flows throughout the project, all headwater and tailwater characteristics, and the measured head loss and generation characteristics of the eight generating facilites. Prior to alternative evaluation, the simulation module was extensively benchmarked to actual production until good agreement was obtained.

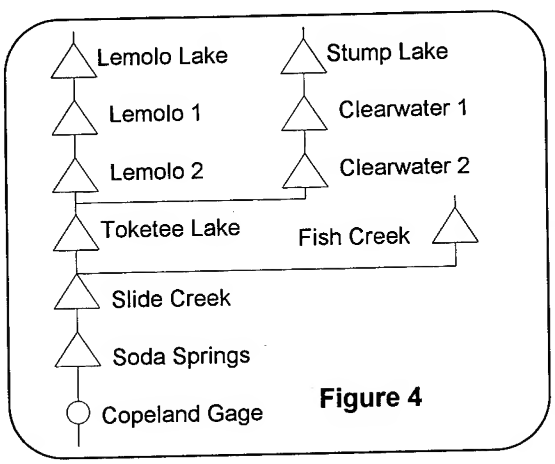

Figure 4

Studies Performed

As of the date of this paper, WaterWay has been used in the evaluation of 46 alternatives being considered within the context of this study of the North Umpqua project. The general nature of these alternatives is summarized in Table 2.

Table 2
North Umpqua Alternatives

Catagory	Number of Alternatives Evaluated
new diversions	13
storage increases	7
storage additions	12
pumped storage	2
upgrade & rehabilitation options	8
additional generation	4

To illustrate the analysis that is being conducted with WaterWay, the effect of the ramprate constraint currently present at the Copeland gage approximately 0.7 miles below the Soda Springs development is presented. This ramp rate constraint is stated as a limitation on river fluctuation that may take place at the Copeland gage. The practical effect of the constraint is to require the Soda Springs facility to operate in a baseload mode, rather than peaking mode. This results in a transfer of energy production from the on-peak to off-peak periods and due to required turbine loading levels, some actual loss of generation.

Figure 5 shows the aggregate hourly project generation for a typical week during the benchmark period. The reduction in peaking operation is quite evident.

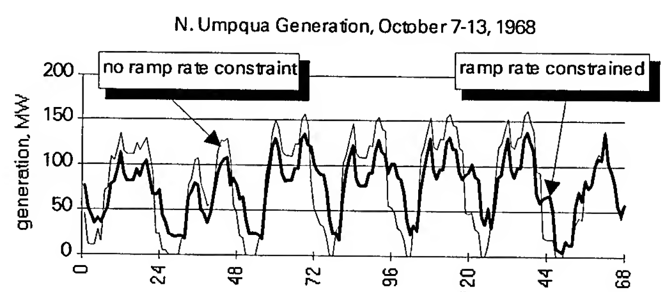

Figure 5

Table 3 develops the change in value of the project generation that results from different peaking operations using WaterWay's simulation mode. This information is useful for several purposes. First, in some cases ramp rate constraints can be met by constructing re-regulating or other storage facilities. The value of peaking operation can be identified and used in balance with the capital and operating cost of such a facility. Secondly, optimized results, improved forecasting, flow measuring systems, and scheduing techniques can reduce the impact of this type of constraint. WaterWay's optimized mode is designed to address these types of measures. The increase in project value can be constrasted with the cost of such measures and economic decisions may be reached.

Table 3
Summary of October 7-13, 1968 Results

Mode	*Ramp Rate Constrained*	*On-Peak Generation (GWh)*	*Off-Peak Generation (GWh)*	*Value* ($1000)*
Simulation	No	10.6	2.4	367.4
Simulation	Yes	8.3	4.5	339.3

* using illustrative values of 30 mils/kwh on-peak and 20 mils/kwh off-peak

Conclusions

Hydroelectric project evaluation in the context of electric system operations may be successfull performed. Many improvements and operational changes impact the operation of the overall power system and require detailed hourly analysis to be adequately assessed. The authors have successfully applied the newly developed WaterWay model for these types of studies on a complex hydroelectric system.

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Biographies

Mark I. Henwood received his BSEE and MSME from the University of California at Davis in 1976 and 1978 respectively. He is the president of Henwood Energy Services, Inc. a consulting firm which performs electrical and hydro system analytical studies.

David K. Hoffman attended Columbia University specializing in computer science and subsequently attended the Artificial Intelligence Laboratory at Stanford University. Mr. Hoffman is the Chief Programmer for The Simulation Group.

Roger L. Raeburn, P.E. received his BSCE from Oregon State University in 1977. Mr. Raeburn is a registered engineer in the states of Montana, Oregon, and Wyoming. He is currently the Manager of Hydroelectric Engineering at PacifiCorp.

Hydropeaking versus Recreation and Environment - *The Power Economic Impacts of the Glen Canyon Trade-off*

By Leslie Buttorff[1], Michael Roluti[2], and Edmund Barbour[3]

Abstract

There is growing national concern over the effect of fluctuating flows below major federal dams where there is competition between existing hydropower economic values and potential recreation and related environmental economic values. This paper reports on the results of comprehensive power system studies being conducted by the Power Resources Committee (PRC) led by the Bureau of Reclamation. This unique committee included active participation by the Environmental Defense Fund (EDF), Colorado River Energy Distributors Association (CREDA), and Western Area Power Administration (Western), with Stone & Webster Management Consultants, Inc.(Stone & Webster) retained to prepare the report and conduct the power modeling studies. A separate investigation addresses the economic impacts related to the renowned Grand Canyon float trips, fishing, cultural aspects, and riverine ecology extending some 300 miles below Glen Canyon Dam. Non-use value studies are also being conducted. The regional power market encompassing over 100 utilities and three million customers will view the results with considerable interest.

The economic studies were designed to identify (1) the effect from the utility viewpoint, (2) the effects from a federal national resource viewpoint, and (3) the impacts on customer rates and power revenues needed for federal project repayment. This paper includes some preliminary values in the PRC draft report of May 1992. Updated PRC values are scheduled to be reported to the WATERPOWER conference in Nashville in August 1993.

Background

There are increasing demands for changing the pattern of downstream releases for hydropower production at existing major federal dams in the U.S. This is primarily due to the negative impacts on recreation, fishery, and other environmental resources. Modification of water power releases will result in the reduction of hydropower economic values in the form of decreased peaking capabilities, decreased total electric energy generated,

[1] Study Manager, Stone & Webster Management Consultants, Inc., 7677 E. Berry, Englewood, CO 80111-2137

[2] Power Business Manager, Bureau of Reclamation, P.O. Box 25007, Denver, CO 80225-0007

[3] Economics Consultant, 2765 So. Golden Way, Denver, CO 80227

or both. Two major river basins are being studied in the Western U.S: the Colorado River with a total federal capacity of about 4,000 MW where the focus is on the 1,400 MW Glen Canyon Dam power plant; and the Columbia River with a total federal capacity of some 20,000 MW, with a number of large federal facilities including Grand Coulee and Bonneville Dam power plants. This paper focuses on the studies led by the Bureau of Reclamation on Glen Canyon Dam. The Corps of Engineers and the Bonneville Power Administration are in the early stages of evaluating the Columbia River.

Some general comparisons between the Colorado River and Columbia River basins may provide WATERPOWER Conference attendees, especially our international participants, with an understanding of the major regional power markets affected. In the case of the Colorado River, the regional power market (Colorado, Utah, Wyoming, Arizona, New Mexico with some interties into Southern California) is predominately thermal-based with limited conventional hydro peaking capacity. The Columbia River market (serving primarily Washington, Oregon, Idaho, and Montana, with some fairly strong interties with California) is predominantly hydro-power-based. It is also "rich" in hydro peaking power and is beginning to experience shortages of firm energy generation. The Glen Canyon Dam water release reductions will primarily affect peaking capability and have little or no effect on total hydro generation. The reverse is true in the Columbia River where firm energy generation will suffer most.

The question in both basins is whether the national economic power benefits foregone are offset by the recreation, fishery, and other environmental economic benefits that are created. These monetary evaluations will provide the basis for comparing the trade-offs in order to enlighten the public and assist in the decision-making process. Other indirect and intangible factors involving multiple objectives[4] of environmental quality, regional development, and social well-being certainly should play a part in the decision process. It will be interesting to follow these pioneering major studies on economic evaluation.

One further introductory point: the number of people affected ultimately enters into the economic equation. In the case of the Colorado River, people directly affected are those who consume the hydropower and those who visit the 300 miles of unique Colorado River Canyon stretching from Glen Canyon Dam to the upper reaches of Lake Mead behind Hoover Dam. The river has limited access and is visited each year by some 60,000 people, principally rafters, fisherman, sightseers, and hikers; white water rafters represent about one quarter of the annual visitors. (Currently the number of white water rafters is restricted and a waiting list is maintained by the National Park Service.) Over two million people visit Lake Powell, which

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[4] For background on multiple objective planning see Barbour, E., "Multi-Objective Water Resource Planning," Second World Congress on Water Resources, International Water Resources Association, December 17, 1975, New Delhi, India.

is formed by Glen Canyon, but little impact on reservoir users is expected. Also, there is expected to be limited impact on the roughly three million people who view the Grand Canyon panorama each year from the canyon rim high above the river, with only limited and occasional views of the river shores far below. Three million hydropower users will also be affected.

Organization of the Study

The Power Resources Committee (PRC)[5] was created in 1989 as an extension of the Glen Canyon Environmental Studies (GCES) authorized in 1982. The PRC's responsibility is to evaluate the economic, project repayment, and rate impacts from changing the magnitude and timing of water releases through the turbine generators.

The GCES effort produced over 40 separate technical studies but excluded the economic and repayment impacts. The PRC economic study results will provide input to the GCES effort as well as to a formal Environmental Impact Statement (EIS) requested by the Secretary of Interior in 1989 and scheduled for public review in 1993. Projected environmental and monitoring studies are expected to amount to about $70 million. The amount of these costs, which are to be paid by Glen Canyon power revenues or from federal funds, is currently being considered.

The PRC reports to an overall economic studies group, as do two other committees placing monetary values on recreation use and "non-use." Led by the Bureau of Reclamation, the PRC is a unique organization that includes active participation by CREDA, EDF and Western, with Stone & Webster retained to prepare large-system modeling studies. As discussed in more detail later, about one-half of the lost peaking power is assignable to "large" systems and the other half to "small" systems. CREDA, which represents the public entities receiving Glen Canyon power, is responsible for initiating studies of small systems. EDF uses Elfin, the Electric Utility Financial and Production Cost Model, to evaluate the large systems. Stone & Webster uses the Electric Power Research Institute's Electric Generation Expansion Model (EGEAS) for the large-systems analysis and is responsible or the overall consolidation and preparation of the PRC report.

A preliminary draft of the initial PRC findings was submitted in May 1992 for internal review. Comments from members of the committee are anticipated to result in significant modifications of preliminary findings and we hope to be able to update the results at the August WATERPOWER Conference in Nashville. Given the make-up of the committee, spirited discussions on basic assumptions, procedures and resulting power values are expected. CREDA represents utilities who have legislative preference to purchase the

[5] This paper reflects the views of the authors, but not necessarily the views of the Power Resources Committee.

relatively low-priced[6] hydro power and will have to pay for lost Glen Canyon power, while EDF is an active national group whose mission is to protect and enhance environmental resources. Western has the responsibility of marketing excess federal hydropower and meeting congressionally mandated project repayment requirements. The Bureau of Reclamation built and operates Glen Canyon Dam and has the responsibility of ensuring that the economic values are objective and reasonably accurate and assisting the Secretary of the Interior in developing permanent operational plans.

Glen Canyon Powerplant Description

The 1400-MW Glen Canyon Powerplant was completed in 1963 and operates with a head of some 560 feet. Average annual flow through the turbines is approximately ten million acre feet, with average generation since its completion of 3.8 billion kWh. Glen Canyon represents nearly 80% of Western's Salt Lake City Area Office resources. Figure 1 shows utilities that directly receive services and that may be affected. Interties with southern California permit some interchanges with the southern California regional power market. The Glen Canyon Dam creates over 25 million acre feet of storage in Lake Powell, primarily to permit the Upper Basin States to use their share of Colorado River water supplies and to meet the delivery requirements of the Lower Basin States and Mexico in accordance with the Colorado River Compact and the Mexican Water Treaty. Hydro power was added to make use of the unique high-dam multiple-purpose site with its valuable generation potential. Power benefits helped demonstrate a favorable benefit-cost ratio supporting economic justification and was the major source of revenues in proving financial feasibility for project repayment.

All revenues go into the Upper Colorado Basin Fund and then are credited among the Upper Basin States in accordance with each state's geographic contributions to water supply. Revenues are used to pay for irrigation and salinity projects within the respective states. The current repayment study extends to the year 2090. Because the powerplant is assumed to have a remaining life of at least 100 years, revenues are expected to continue beyond the payout period. Appropriate replacement provisions are provided for facilities with shorter service lives.

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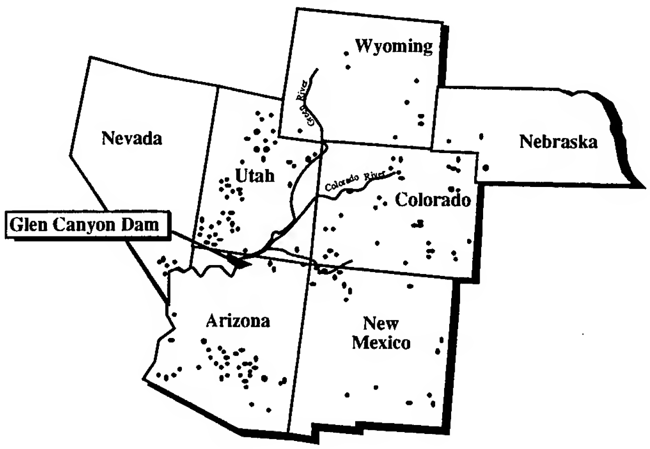
[6] It is "low priced" when compared to the costs of replacement by new generation. The low price stems from congressional authorization where it required that only the relatively low costs of an efficient high hydro site built at 1960 costs and low interest rates be recovered in 50 years plus an additional component for assisting the payout of irrigation and salinity projects in about 90 years. The power is not priced specifically as peaking capacity but as firm power backed up by Western's firming energy purchases.

Figure 1
Map of Potentially Impacted Utilities



The Alternative Studies

Six sets of alternative flow patterns (Table 1) were compared to the base case or no action plan. Three alternatives tested variations in fluctuating releases, and three tested variations in steady flow releases. In the revised study, one additional alternatives is being added: the "maximum powerplant capacity."

The base case represents average conditions before the recent adoption of the interim flow regime (mentioned earlier) which resulted in an average reduction in hydro peaking capacity of about 400 MW. Two marketing arrangements were used to derive two comprehensive sets of values:

- Contract Rate of Delivery (CROD) approach - based on contracted rates of delivery including purchased power energy by Western.

- Hydrology approach - based on reservoir releases reflecting the continuously changing hydrologic cycle of the river and not including firming energy purchases by Western. Non-firm energy purchases and sales were considered, however.

Table 1 - Glen Canyon Dam EIS alternatives

Alternative	Description
STUDIED IN DETAIL:	
No Action	Maintain the existing fluctuating releases (prior to the 1991 modified interim flow releases) and provide a basis for impact comparison.
Maximum Powerplant Capacity	Permit use of full powerplant capacity.
Restricted Fluctuating Flows	
High	Slightly reduce daily fluctuations from historic no-action levels.
Moderate	Moderately reduce daily fluctuations from historic no-action levels.
Low	Substantially reduce daily fluctuations from historic no-action levels.
Steady Flows	
Existing Monthly Volume	Provide steady flows that use existing monthly release strategies
Seasonally Adjusted	Provide steady flows on a seasonal or monthly basis.
Year-Round	Provide steady flows throughout the year.
CONSIDERED BUT ELIMINATED:	
Mimic Predam Flows	Provide flow conditions that are similar to predam conditions; high, warm spring floods and sediment augmentation.
Run-of-the-River	
Historic Pattern	
Reregulated Flow	Maximize fluctuations from the dam to Lees Ferry, with a reregulation dam providing near-steady flows downstream.
Seasonally Adjusted	Moderately reduce daily fluctuations during fall, winter, and spring; substantially reduce daily fluctuations in the summer.

Because of technical problems encountered with the hydrology approach in the initial results, only the CROD values are reported in this paper. Values related to hydrology will be covered in ensuing studies. Table 2 summarizes the capacity impacts in MW (from the May 1992 draft report).

Table 2
Glen Canyon Dam Peaking Capacity Losses - CROD
(MW)

Fluctuating Flow Alternatives	Summer	Winter	Steady Flow Alternatives	Summer	Winter
Low	386	447	Year-Round	686	714
Moderate	372	409	Seasonally Adjusted	764	803
High	24	46	Existing Monthly	579	616

Experienced hydropower planners may reasonably ask if a reregulating dam was considered as a possible alternative solution to the conflict in uses. This would permit baseload hydro energy at the reregulation site and permit optimum releases to meet downstream recreation and environmental needs. A reregulating alternative would inundate some 16 miles of the 300 miles of flowing river and construction activities would affect the ecology of the canyon. Because of these reasons, the EIS plan formulation team considered reregulation not to be a viable alternative for economic evaluation and the resulting data not significant to the general public in the decision-making process.

Study Process

Rather than using a sampling approach, the entire regional power market area that receives Glen Canyon power from the Salt Lake City Area/Integrated Projects was modeled. These projects include: Glen Canyon, Blue Mesa Dams, Crystal, Flaming Gorge, Fontenelle, and Morrow Point. As mentioned earlier, the utility systems were analyzed in two categories:

- Large systems, which represent about one-half of the power market and typically own generation resources, were analyzed by applying the two sophisticated simulation models, EGEAS and Elfin.

- Small systems, which typically do not own generation resources, were analyzed using a simpler spreadsheet model that recognized surveyed costs of purchasing replacement power.

The large systems - covering six separate systems with their associated interconnections - were modeled independently. The utilities were not combined into a single entity because they are non-centrally dispatched and transmission constraints do exist. Inter-utility transactions were quantified separately by setting up the utilities as interconnected systems. For

example, a utility interconnected with three surrounding utilities was modeled in this manner. The base case was first simulated and then compared with the six alternatives. The differences were derived for each alternative as a basis for determining the economic impacts.

CREDA conducted a survey to determine the economic value to small systems. The data were organized into spreadsheets by utilities and states and summarized for the total small-system values. Table 3 lists the utility members of the large and small system analyses and the winter capacity allocations. These amounts reflect the total power received before peaking capacity losses; they do not reflect capacity losses attributed to alternative flow alternatives.

Utility and Federal Viewpoints

The results of the PRC economic studies are to reflect both the utility viewpoint and the federal viewpoint. The same interest rate (8.5%) and projected price escalation rates are reflected in both viewpoints. The federal viewpoint is to be further adjusted to remove "transfer payments" in interutility transactions. (In economic theory, transfer payments are transactions that do not add national income, but simply represent dollars shifted from one pocket to another. A classic textbook example is welfare payments that simply transfer dollars from one income group to another, resulting in no change in overall national income.) The application of this sophisticated concept to electric utility economics resulted in differences of opinion on how this principle is implemented. These differences are to be reconciled in the final report and the effect can be appraised by comparing the results of the utility values and the lower federal values. The May 1992 draft attempted to address this issue. Preliminary results discussed later in this paper will be limited to the economic values from the utility viewpoint, since this is the approach most familiar to conference participants and generally not subject to controversy except as discussed later with regard to the treatment of excess generation resources and conservation measures.

The May 1992 draft also included a federal-type constant-dollar approach used by federal water agencies in evaluating new hydropower resources. That approach eliminates general inflation in computing the replacement costs of short-lived replacement plants (such as 20-year combustion turbines) as well as in estimating operating and fuel expenses. The basic federal assumption made in using the constant-dollar approach in developing benefit-cost ratios for new projects is to simplify the analysis for public and congressional review. There is also the implied assumption that applying general inflation equally to both sides of the benefit-cost equation would normally be offsetting. The adjustments to a federal constant-dollar approach in the May 1992 draft indicated a general reduction in the magnitude of the utility viewpoint values by about one-half. The revised draft will recognize inflation in the federal viewpoint analysis and for information purposes will include the effects of a constant-dollar approach in supporting data.

Table 3
Large and Small Systems Analysis
SLCA/IP Members

Large System Analysis		Small System Analysis			
Large Utilities	SLCA/IP Winter Allocation (MW)	Small Utilities	SLCA/IP Winter Allocation (MW)	Small Utilities	SLCA/IP Winter Allocation (MW)
Salt River Project	52.113	**ARIZONA**		**NEVADA**	
Arizona Power Pooling Association	15.408	Ak-Chin	1.92	Valley Elect Assoc	8.876
Deseret	134.431	Chandler Heights	0.302	Overton Power Dist	8.876
Tri-State G&T	226.027	CRIR	0.881	Boulder City	7.827
Colorado Ute	56.002	ED-3	2.88	Pacific Engineering	3.896
Colorado Springs	64.864	ED-4	3.68	Deseret	16.902
Platte River Power Authority	145.955	ED-5 M	0.233	**NEW MEXICO**	
		ED-5 P	2.633	Aztec	2.778
		ED-6	0	Cannon AFB	1.419
		ED-7	0.729	Central Valley Elec	3.081
		MWD	2.372	County of Los Alamos	1.569
		NTUA	23.675	DOE (Alb Oper Off)	36.126
		Octillo WCD	0.272	Farmers Elec Co-Op	2.353
		Queen Creek ID	0	Farmington	18.866
		Roosevelt ID	1.761	Gallup	3.592
		Roosevelt WCD	1.616	Holloman AFB	2.065
		Safford	0.56	Lea County Elect	2.335
		SCIP	1.84	Plains G&T	177.717
		San Tan	0	Raton	1.637
		Thatcher	0.363	Roosevelt Co Elect	2.517
		W-M IDD	0.448	Sandia/Kirtland AFB	3.592
		Williams AFB	0.912	Truth or Consequence	5.506
		YPG	0.415	**UTAH**	
		COLORADO		Brigham City Corp	12.594
		Aspen	1.677	Def Dept, Ogden	3.532
		Center	1.801	Helper	0.472
		Delta	1.721	Hill AFB	3.592
		Fleming	0.068	ICPA	206.632
		Fort Morgan	9.08	Deseret	44.149
		Frederick	0.045	Kanab	0.611
		Glenwood Springs	1.689	Levan	0.467
		Gunnison	7.224	Moon Lake EA	57.644
		Haxtun	0.546	Manti	1.866
		Holyoke	2.023	Nephi	3.975
		Lamar Util Board	2.662	Price	1.702
		Northern Colo WCD	0	Provo	72.455
		Oak Creek	0.485	Salem	0.851
		Pueblo Army Depot	2.856	Spanish Fork	6.24
		Willwood L&P Co	0.039	Tooele Army Depot	1.307
		Wray	1.059	Univ of Utah	3.461
		Yuma	1.411	Utah State Univ	1.152
				WYOMING	
				Bridger Valley EA	10.558
				Torrington	1.302
				WMPA	6.731

Conceptual Issues

Two major conceptual issues were raised in addition to the "transfer payments" question mentioned previously: (1) whether to count the economic fixed capital cost of resources used in replacing lost Glen Canyon capacity when there are excess reserves of capacity in the individual systems being modeled (this reflects the use of standby existing coal-fired thermal plants, nuclear plants, or combustion turbines) and (2) whether low-cost conservation (primarily demand-side management (DSM)) is a proper

surrogate value for an existing hydro production facility that currently generates power on peak. These are most significant questions in measuring economic values occurring early in the 50-year period of analysis. From a practical standpoint, a future DSM resource can rarely compete with an existing hydropower resource because of hydropower's extremely low operating cost (no fuel); however, it often can compete with an existing coal-fired resource because of the relatively high operating costs. Current utility studies indicate that existing hydro resources are the very last to be displaced by DSM.

For some of the large systems modeled, the use of excess reserves (either within the utility or purchased from other utilities) can occur in the first two years, five years, ten years, or theoretically for the entire 20-year planning period. When recognizing the effects of discounting at the assumed interest rate of 8.5%, the essentially "free" economic use of excess reserves (counting only minimal variables costs) can reduce levelized values over the 50-year period of economic analysis significantly (about 1/3 for a five-year period, 2/3 for a ten-year period, and 4/5 for a 20-year period).

With regard to the use of existing excess reserves as a substitute for the existing Glen Canyon peaking capacity, some economists argue that only the variable out-of-pocket operating costs associated with the electric production machinery in the reserve inventory should be counted as an economic cost. Others argue that both fixed capital costs and variable costs related to the production machinery substituting for Glen Canyon should be counted as "true" economic costs. The use of only variable energy costs drastically reduces the values attributable to Glen Canyon. Since there is no change in overall system generation with the loss of Glen Canyon peaking, but simply a shift from producing one kWh on peak to producing one kWh off-peak, the resulting minor differences in variable costs relate either to a difference in fuel costs if there is a shift from gas to coal (it could also mean coal to coal) or to changes in heat rate efficiencies. Thus, the impacts will be quite different depending on the snapshot of years selected for the analysis. Nevertheless, this approach, which reports only variable costs when excess reserves were available, was used in the study to determine the economic values for the large systems.

Other economists argue that fixed capital costs consist of costs components for existing resources designated for recovery of the initial investment and the cost of money; in other words, to provide for the depreciation or "using up" of the production facility over its service life. Obviously when a machine in the inventory of reserves that has a ten-year residual service life is used for peaking for five or ten years, a significant share or all of its remaining useful life may be consumed. This can be viewed as an economic cost to the utility as well as to the nation.

The EGEAS model, used to measure the large-system impacts, is designed as a planning tool to develop expansion plans for <u>future</u> resources (both supply and demand side) and to optimize the operational costs of existing

and future resources. The model does not usually include the fixed capital costs of existing capacity regardless of whether excess reserves exist. EGEAS is designed to assist the planner in minimizing future out-of-pocket operating costs of existing plants and to provide guidance for adding new investments as needed to meet future loads. Provisions can be made in the EGEAS model to account for the fixed capital cost component of Glen Canyon replacement peaking capacity whether the excess capacity is owned by the utility being modeled or comes from the inventory of another utility. The ELFIN model, also used in the large-system analysis, is principally a production costing model that optimizes operating costs directly, but indirectly deals with capacity-fixed capital costs outside the model. The EGEAS future capacity expansion plan is fed into the ELFIN model to determine the future capacity costs.

In an overall system analysis, the inclusion of demand-side management can reduce the marginal cost effects attributed to the exclusion of Glen Canyon peaking capacity. DSM peak-shaving measures to reduce a future need for peaking capacity have a levelized cost of about $40 per kW year, while the levelized cost of a combustion turbine in a 34-MW size is $150 per kW year assuming no net change in total system generation. The variable operating costs for the CT are limited to the difference between the cost of gas and coal. (It assumes that the CT would generate 400 hours per year on peak, which would offset 400 hours of coal-fired generation off-peak---from the PRC May 1992 draft data.)

Resolution of these two major issues mostly affects the large systems; however, it would affect the small systems if fixed capital costs of excess resources are omitted from computed avoided costs or purchased replacement capacity during the early years when the utility from which they purchase capacity has excess resources. Also, the small-system values would be significantly lower if DSM costs are used as a substitute for Glen Canyon peaking.

Preliminary Comparisons of Available Results

We mentioned earlier that the preliminary results included in the May 1992 PRC are being updated. For two of the fluctuating flow alternatives -- moderate and low -- where 410 to 450 MW of Glen Canyon peaking capacity were lost, the utility viewpoint levelized economic values amounted to about $80 million annually for each alternative when effects for both the large and small systems are combined. The three steady-flow alternatives resulted in annual peaking capacity losses from $100 to $130 million. These numbers reflected the CROD approach assuming firming purchases by Western.

The breakdown between the large and small systems disclosed a wide disparity in economic values even though each of the large and small systems bare approximately half of the total capacity loss. These differences can be better understood when converted to average levelized

cost per kW year: the large systems averaged about $40 per kW year while the small systems averaged of $280 per kW year.

The major difference was due to the assumption regarding the omission of fixed capital costs on excess resources and relatively low DSM costs used in Glen Canyon in the large-system analysis. These factors significantly affected the early year values for several systems modeled. The small-system values accounted for fixed capital costs in the early years of analysis, whether the replacement peaking power purchased reflected existing plants in reserve or new plants in the future. Future DSM measures were not considered an appropriate substitute for the replacement of existing Glen Canyon peaking power in the small system analysis.

The only published data available on the value of Glen Canyon peaking losses for comparison at this writing (January 1993) are presented in the EDF report "Conflict on the Colorado River" (August 1992). That report states that the "short-run" costs are $6-8 million per year for apparently the loss of 400 MW, cited as the effect of interim operations. This amounts to $15 to $20 per kW year as the economic value of Glen Canyon, which "is due to the current surplus of generating capacity in the region." This compares to the short-run values developed by CREDA for the PRC May report which averaged $200 per kW year and which would amount to an average of $80 million a year for the first five years. The EDF values are from its ELFIN runs. The EDF also cites a "worst case" scenario of building combustion turbines today to replace the 400 MW lost at a cost of about $20 million annually or $50 per kW year. When recognizing inflation of future plant replacement and operating expenses, the PRC May 1992 report showed a levelized cost of $150 per kW year for new combustion turbines, reflecting its 20-year service life during the 50-year period of analysis. For 400 MW, this amounted to $60 million per year.

Repayment Studies

The preliminary repayment studies developed by Western for the CROD analysis indicated increases in average composite rates from 19 mills per kWh in the base case, to 23 to 29 mills per kWh for a range of losses in peaking capacity from 410 MW to 800 MW. A rough rule of thumb derived from the PRC report suggests that a one-mill rate increase produces about $6 million in average annual revenues. In the early period of analysis, this would result in related total revenue losses of $24 million to $60 million. No attempt was made to correlate these rates to levelized losses in revenues comparable to the levelized values from the utility viewpoint. Also, peaking capacity components were not separately identified. The payout period extended through 2090. The marketable resource was held constant with purchases made to make up Glen Canyon peaking losses. Further analysis and identification of levelized total annual revenues lost are expected in the final draft. How to treat inflation for replacement and purchased energy is another procedural problem to be resolved.

Conclusions

The Glen Canyon studies on the economic losses of hydropeaking are plowing new ground on a subject that will receive growing attention in the major river basins in the Western U.S. A number of conceptual issues remain to be resolved, however, and these issues can have significant effects on the economic values that measure the trade-offs of reduced hydro peaking. The two major conceptual problems occur: (1) where the use of excess existing reserves as a replacement for hydro peaking capacity carries relatively low economic costs by being limited to variable out-of-pocket system costs (excluding depreciation and the cost of money related to the construction of excess generating plants), and (2) the use of relatively low costs of future DSM programs in system capacity studies, as a surrogate value for existing hydropower peaking capacity.

Wind/Pumped-Hydro Integration And Test

Warren S. Bollmeier II,[1] Ning Huang[2] and Andrew R. Trenka[3]

ABSTRACT

The State of Hawaii (SOH) is supporting the enhanced use of renewables as a means to reduce the State's dependence on petroleum and to create new market opportunities that will stimulate both the U. S. renewable energy industry and the economy of Hawaii. As wind and other renewable technologies continue to mature, opportunities for their application in Hawaii increase. However, in Hawaii the penetration of windpower has been limited partly due its intermittent and fluctuating nature. The penetration of wind-power could be increased if it can be "smoothed" or "firmed" via the addition of a storage component, such as pumped-hydro. Therefore, the project entitled "Wind/Pumped-Hydro Integration and Test (WPHIT)" was initiated by the Pacific Center for High Technology Research (PICHTR) in April 1992 at the Renewable Energy Storage and Test (REST) Facility on the Kahua Ranch, Island of Hawaii, in cooperation with the SOH's Department of Business, Economic Development and Tourism (DBEDT), Hawaiian Electric Light Company (HELCO), Hawaii Natural Energy Institute (HNEI) and Kahua Ranch Limited (KRL). The specific objectives are to develop and utilize the REST Facility to: simulate utility-scale applications, investigate alternative system architectures, operational modes and control strategies, and demonstrate power smoothing and firm power delivery. Component test results are presented along with plans for preliminary system tests and the WPHIT conceptual design.

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1 Manager, WPHIT Project, Pacific International Center for High Technology Research, 2800 Woodlawn Drive, Suite 280, Honolulu, Hawaii.

2 Project Engineer, WPHIT Project, Pacific International Center for High Technology Research, 2800 Woodlawn Drive, Suite 280, Honolulu, Hawaii.

3 Vice-President Operations, Pacific International Center for High Technology Research, 2800 Woodlawn Drive, Suite 280, Honolulu, Hawaii.

The State of Hawaii (SOH) is supporting the enhanced use of renewables as a means to reduce the State's current dependence on petroleum and to create new market opportunities that will stimulate both the U. S. renewable energy industry and the economy of Hawaii.

PICHTR, in collaboration with the SOH Department of Business, Economic Development and Tourism (DBEDT), the Hawaii Natural Energy Institute (HNEI) of the University of Hawaii, and the Kahua Ranch Ltd. (KRL), is developing the Renewable Energy and Storage (REST) Facility, at Kahua Ranch on the Island of Hawaii to field test and evaluate renewable energy and energy storage technologies, including wind, PV, pumped-hydro, battery and hydrogen components.

REST Facility Capabilities

The REST is a unique facility for research, development and demonstration of integrated renewable energy generation and storage technologies. Kahua Ranch is an excellent wind site with annual average windspeeds ranging from 7.5 to 11 m/s at many locations on the ranch. The REST Facility has the following major installed components:

- Five wind turbines for electrical energy production: total capacity of 102.5 kW [three 25 kW Carters (480VAC/3Ø), one 17.5 kW Jacobs (240VAC/1Ø) and one 10 kW Jacobs (240VAC/1Ø];

- A Pumped-Hydro Energy Storage (PHES) System: ~276 kWh capacity (usable water capacity of 33.38 m^3 (529,000 gallons), 1006 m (3300') long, 0.2 m (8 in) diameter penstock, 101 m (330')head, and a 50 hp reversible, turbine-pump assembly);

- A Battery Energy Storage Bank (BESB): 142 kWh capacity (1185 Amp-Hours at 120VDC) with 60 gel-type, lead-acid cells and a 9 kW converter;

- A diesel-electric generator: rated capacity of 135 kW at 480VAC/1Ø;

- Four metal hydride storage systems: total hydrogen storage capacity of ~180 Nm^3, or equivalent electrical energy storage of 540 kWh; and

- Wind and solar data measurement systems (including a 27.4 (90') and 36.6 m (120') meteorological towers).

REST Facility Objectives and Near Term Work

The objective of the research at the REST Facility is to investigate the commercial viability of integrated renewable energy and storage systems utility-scale power generation in Hawaii. The REST Facility is tailored to be a field simulator for larger hybrid systems for utility application. At the Facility, different system architectures, operational modes, and alternative control strategies will be investigated. The first project underway is the Wind Pumped-Hydro Integration Tests (WPHIT). It focuses on the component characterization tests, development of master controller, and integrated system operations for power generation with improved quality.

WPHIT PROJECT OVERVIEW

As wind and other renewable technologies mature and become more cost effective, opportunities for their application in Hawaii increase. However, in Hawaii and other island utility environments, the penetration of windpower has been limited partly due to its intermittent and fluctuating nature. The penetration of windpower could be increased in Hawaii if it can be "smoothed" or "firmed." Addition of a storage component, such as pumped-hydro, with windpower is proposed as one possible solution. Therefore, the project entitled "Wind/Pumped-Hydro Integration and Test (WPHIT)" and described herein, was proposed for the evaluation and demonstration of the technical and economic feasibility of integrated wind/pumped-hydro systems for "firm" delivery of electricity to the utility. The first year of WPHIT is underway at the Renewable Energy Storage and Test (REST) Facility on the Kahua Ranch of the Island of Hawaii with a total of $225K in funding.

Goals

The overall objective of WPHIT project is to investigate the commercial viability of integrated wind/pumped-hydro systems for firm, utility-scale power generation in Hawaii. The specific technical objectives are to:

- Develop and utilize the REST facility as a simulator for larger systems for utility application,
- Investigate alternative system architectures, operational modes, and alternatives control strategies, and
- Demonstrate power smoothing and firm power delivery with operational characteristics acceptable to the utility.

Scope

The first year's effort will focus on:

- limited site modifications to enhance the existing facilities, including an improved data acquisition system and utility interconnection;
- preparation of a WPHIT conceptual design;
- component characterization tests; and
- preliminary system tests.

Preliminary Component Test Results

The primary objectives of the component test are to characterize the overall performance of each of the primary components under consideration for the WPHIT. This characterization includes measurement of power and energy outputs, power factors, mechanical and electrical efficiencies, and availabilities, and factors important for system integration, such as control functions (ramp rates and turnaround times), power fluctuations, power availability and wind resource data. and are the wind turbines, the pumped-hydro energy storage (PHES) system, and the battery energy storage bank (BESB). Prior to WPHIT, the Hawaii Natural Energy Institute (HNEI) collected wind data and installed, operated and tested the three J. Carter wind turbines, the PHES and the BESB. The wind turbines have been operated since their installation, one each in 1988, 1989 and 1990. General performance and operation and maintenance were collected. However, a standard power performance curve have not been obtained as a met is not currently located near the turbines. Future plans call for installation of an additional met tower just upwind of the three Carter wind turbines. The PHES installation was completed in 1992. The initial tests performed by HNEI confirmed a peak generating output of 25 kW at 52.5 kg/s (814 gpm), and a fixed pumping rate of 14.7 kg/s (228 gpm) at 27.3 kW. In addition, preliminary tests of a variable speed controller for the PHES were conducted by HNEI with funding from the Bonneville Power Administration (BPA). These tests were less successful and were terminated after repeated thyristor failures. The unit was returned to the manufacturer for diagnostics and evaluation. No further tests are planned at the present time. The BESB was installed in 1986. HNEI verified the capacity of the bank via several charge and discharge cycle tests. Since that time, the bank has not been operated on a regular basis.

Under WPHIT, a centralized data acquisition and control system (CDACS) system has been established which is used to monitor wind speed/direction, component performance and the electrical system characteristics on the REST Facility. The wind data are currently monitored on the 27.4 m (90') met tower which is located near the PHES pumphouse. The key component performance parameters are sampled via a Campbell Scientific datalogger (Model CR-21X) every two seconds and are pre-averaged to produce 10-minute samples. These data are stored temporarily in the datalogger, then downloaded to a PC. The electrical system parameters include overall system frequency and voltages on the three primary buses: a 240VAC bus (includes the office, workshop, small greenhouse and two Jacobs wind turbines), a second 240VAC bus (the Kahua Ranch) and 480 VAC bus (includes the three J. Carter wind turbines, the PHES and the large greenhouse). A net power metering point is currently being installed to allow monitoring of the net power flows in/out of the facility. The component tests have confirmed the response rates and characteristics of both the PHES and BESB. Finally, a year's worth of wind data, which were collected at the 37.7 m (120') elevation, were reduced and analyzed. The results indicated an annual average wind speed of 10.0 m/s (23.4 mph) with consistent winds throughout the year.

Preliminary System Tests

Preliminary system tests will be performed in a manual control mode. The tests will be initiated with the WPHIT configured with the three J. Carter wind turbine, the PHES and the BESB. The system (master) control function will be provided by two project team members. The system will be operated in two control modes: first, for power smoothing and, second, for peak power delivery. The results of these tests will then be used to evaluate the WPHIT conceptual design and to prepare the preliminary specifications for the automatic master controller, which is to be developed as part of the second year's activities. Once the master controller has been installed and checked out the system will be tested in an automatic mode for extended periods to demonstrate power smoothing, peak power delivery and possibly baseload power delivery.

Methods for Appraising International Projects

George K. Lagassa, PhD[1]

Abstract

Several major, multi-use dam projects in less developed countries have come under fire recently from non-governmental organizations for inadequate attention to their broader, negative impacts on the natural environment and on surrounding economies, societies, and cultures. Appraisal of these projects requires an a comprehensive analytical approach for weighing all of the tangible and intangible, first order and second order, costs, benefits, and other results associated with such projects. Quantitative precision is impossible for every aspect of such a comprehensive approach. Some aspects can be valued by technical routines, others require judgement and political decisions. The best appraisal methods should be designed to be comprehensive, to increase conceptual clarity, and to distinguish between those elements where precision is possible and those where it is not.

Introduction

Anyone who has been involved in hydroelectric licensing in the United States is familiar with the increasing complexity of the process for evaluating proposed hydroelectric projects. Gone are the days when routine cost-benefit analysis provided the basis for rational decision making. Today, rationality involves the explicit consideration of numerous values in addition to, and often in conflict with, the benefits of additional electrical energy and ancillary recreational benefits. Indeed, the complexity may be

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[1]Licensed Commercial Appraiser, Principal, Mainstream Associates, 26 Maple Road, P.O. Box 947, North Hampton, NH 03862.

so high as realistically to prohibit the prior identification of totally rational and optimum choices.

Yet the complexity of these considerations in the United States setting pales by comparison to the complexity of hydroelectric decision making in international settings. The reasons for this are twofold and related. First, in international settings, the possibility still exists of siting a large dam, while in the United States the era of large dam construction is over. The last large hydroelectric dam constructed in the USA, the Glen Canyon Dam in Arizona, was completed in 1966, a few years before the enactment of the National Environmental Policy Act, which symbolizes the birth, for real, of the modern environmental protection movement. As a result, the opportunity to weigh the mega-impacts (cost and benefits) of such mega-projects simply no longer arises in the United States. In addition, the choice between single large projects and multiple smaller ones is moot.

Second, the vast majority of large dams still being built are sited in less developed nations. Faced with minimal domestic product, low agricultural productivity, and rapid population growth, many of these nations are anxious to host major, multiple use dam projects as a vehicle for modernization and a direct path to the first world. However, because these projects are largely financed by first world lenders, they are influenced increasingly by first world thinking and decision making processes. The 1991 Pelosi Amendment, for example, requires the U. S. Directors of all multilateral banks to vote no or to abstain from any lending decisions with respect to which the director did not receive a full-blown Environmental Impact Assessment 120 days in advance. In effect, the lender is expected to make optimum choices with respect to numerous public policy variables well beyond mere economic feasibility.

The extent to which such decision making requirements will broaden the scope of pre-construction analysis is not yet known. However, recently completed independent reviews of both the Sardar Sarovar projects in India and the Three Gorges project in China suggest parameters which are extremely broad, involving the clarification and analysis of fundamental (and complex!) political choices about paths to economic development and modernization.

In this paper, we will examine the limits of existing approaches to the appraisal of hydroelectric development projects in international settings and show the outlines, and the weaknesses, of the more comprehensive decision making model which appears to be required.

MARKET VALUE: THE APPRAISERS' APPROACH

In the United States, lending decisions for private sector hydroelectric projects are typically based on a project valuation prepared by a professional appraiser. Using the three standard techniques of real estate appraisal -- sales comparison, replacement cost analysis, and income capitalization -- the professional appraiser is able to report a market value for a proposed project, and the lender then determines whether this value offers sufficient collateral and debt coverage to merit the investment risk.

For the professional appraiser, market value is understood to be "the most probable price in cash, terms equivalent to cash, or in other precisely revealed terms, for which the appraised property will sell in a competitive market under all conditions requisite to a fair sale, with the buyer and seller acting prudently, knowledgeably, and for self-interest, and assuming that neither is under undue duress."[2] This definition of market value will incorporate environmental concerns and values only to the extent that a transaction takes place which involves environmental amenities -- i. e., project capital and/or operating expenses are allocated to the mitigation of environmental impacts. If the incurred costs of mitigating project impacts do not increase total project costs or operating expenses above a level which reduces debt coverage ratios or investment returns below acceptable levels determined by the lender, then the project goes forward.

Both the strength and the weakness of market valuation as a basis for project decision making is that it presumes the existence of a market in which exchange transactions occur freely and establish market perceptions of value. Where a market exists, a professional appraiser can determine with some

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[2] American Institute of Real Estate Appraisers, *The Dictionary of Real Estate Appraisal* (Chicago, 1984, 194 - 195.

precision, the market's perception of value. The appraiser adds up the numbers; and the lender, by establishing threshold levels of acceptability for coverage ratios and investment returns, evaluates the sums. Depending upon how the numbers add up, if the coverage ratios and investment returns are achieved, then the decision is obvious. Neither the appraiser nor the lender/decision-maker is asked to value externalities which are not involved in a market transaction -- that is, external costs which go unpaid, or external benefits which go unrewarded. The appraiser's sole intention is to identify market perceptions of value, not to allocate any special value to environmental amenities. Likewise the lender defines only the acceptable threshold level of returns and debt coverages.

Significantly, this decision making approach totally separates the appraisal activity from the lending decision, a luxury which is not possible for lending decisions about international projects by multilateral banks. In effect, the Pelosi Amendment requires the lender to integrate the investment decision -- more typically made on the basis of objective and quantifiable, market-based values -- with consideration of additional environmental amenities for which no identifiable market exists. Therefore, the appraisers' approach to project valuation, taken alone, is inadequate to support international lending decisions. Moreover, there does not exist a market for large dam projects in any case, since they are usually so large as to require government ownership and development.

BENEFIT COST ANALYSIS: THE ECONOMISTS' APPROACH

Because environmental amenities typically are not bought or sold on the open market, it is impossible to assign to them a specific, measurable value. Economists have attempted to overcome this problem by identifying implicit market transactions and assigning proxy values to the ancillary benefits of hydroelectric projects. Thus, for example, the benefit of scheduled water releases for white water boaters may be measured by the amount of money which potential users would be willing to spend in order to enjoy the sport of white water boating. The precise value contained in this implicit market transaction is measured through survey research of an identified audience of likely users. This research may in turn be supplemented by an analysis of the multiplier effect in the local or

regional economy of the expenditure of tourist dollars on the sport of white water boating.

The costs of providing or protecting each of the ancillary environmental or recreational benefits at any particular hydroelectric project are considerably easier to measure, since this involves a market transaction -- i.e., the spending of dollars on increased capital or operating expenses and/or the opportunity cost of foregone revenues. So long as the cumulated benefits exceed the cumulated costs of any particular project, it is given a benefit cost ratio greater than 1 and thus qualifies for a "go" decision.

The measurement of ancillary benefits by means of such proxy measures can be of questionable accuracy. Fo example, efforts have been made to measure the benefits of the Atlantic Salmon Restoration Program in terms of the dollar benefits of the number of "angler days" which sport fishermen will likely devote to the act of fishing. By this analysis, the more time devoted to catching fish, the higher the benefit, which makes possible the unintended conclusion that sport fishing benefits will increase if we limit the population of sport fish, as opposed to increase it.

Such distortions notwithstanding, this level of accuracy and precision is probably the best that can be hoped for in our efforts to impose quantitative decision making techniques on such value-laden, qualitative decisions. The problem becomes even worse when we begin to factor in the increasing interrelationships among variables necessitated by basin-wide, cumulative impact assessments.

In the United States, we have attempted to face this complexity through the regulatory process, not in the lending process. Here, the effort to assign value to environmental amenities associated with hydroelectric projects precedes the lending decision and is undertaken in the licensing process -- a process which may be viewed as supplemental to the appraiser's task. Thus, the appraiser's identification of market value (and, implicitly, of economic feasibility) is passed on' to the lender with an additional valuation of external costs and benefits. Since project financing simply cannot proceed without a project license in hand, it is fair to say that the lender's decision presumes all environmental costs and benefits already have been considered. Given the Pelosi amendment, this is a luxury not offered to multilateral lending

institutions which are considering international projects.

Thus, the international lender is faced with an extremely difficult situation. In order to make a lending decision, the required level of analysis is highly complex, the possible level of precision is quite low, and the potential for error is very high. To complicate matters further, we can be sure that the analytical process will be monitored by numerous interested parties, including selfish private interest groups (whose values may be both measurable and clear) as well as groups whose interests lie in achieving a particular definition of the public good. Any perceived "error" will most certainly be noted and announced publically and alternative analyses presented to bring into question the lenders' conclusions and analytical support. What is a lender to do?

THE NEED FOR SYSTEMS THINKING

It must be tempting for lenders to throw up their hands and arrive at the cynical conclusion that all such decisions are political, and not subject to rational analysis. While it is true that significant portions of the analysis are unable to be conducted as mere technical exercises, lenders can still attain to conceptual clarity and try to take a comprehensive approach to their decision-making about proposed international hydroelectric projects. To this end, it is important to identify what elements of the pre-funding analysis are and are not amenable to technical evaluation.

A completely rationalized decision-making process would require analysis first and foremost of the highest political goals, since only after we have identified goals can we define the proper means for achieving them. However, we would argue that it is not the responsibility of lenders to dictate the broad political goals of less developed nations. Rather, public goals are a matter of public policy which will emerge from political processes that are influenced by numerous variables extraneous to the lenders' concerns: democracy, pluralism, tradition, oppression, and cultural values. Therefore, we believe it is best to assume that broad socio/economic goals are a given.

It is probably useful to think of the project decision as a system set within a given, two tiered, environment. The impact of the system on the

environment, and of the environment on the system, is reciprocal. The extent of the lender's analysis, rightfully, can only extend to the reciprocal relationship between the first order environment and the system. Impacts on and of the second order environment are not subject to technical evaluation.

The starting point for the lender has to be a specific proposed project of an identified size, scale and purpose. Typically, the larger dam projects are multi-purpose projects designed to serve all or several of the following objectives: (1) irrigation for agricultural purposes, (2) improved drinking water quality and quantity (for public health purposes), (3) flood control (for commercial stability and public safety purposes), (4) increased electric energy supply (for industrial, commercial and or residential uses), and (5) improved navigation (for commercial development and overall infrastructure improvements).

With these goals established it is reasonable to ask if there are any other, possibly superior, means of achieving the same results. For example, a 500 MW hydroelectric project may be unnecessary, indeed totally inappropriate, in a nation with average annual load growth on the order of 25 - 50 MW. In such a setting a much smaller 50 MW project, requiring a far smaller financial commitment, would probably be more appropriate, if that is the only goal being served. However, a smaller project, with a proportionately smaller impoundment may be less capable of offering significant flood control, drinking water supply and improved navigation; thus making inappropriate the choice of a substantially downscaled project.

On the output side, measurement of first order costs and first order benefits from such projects is not always easy, but it is probably subject to technical evaluation. **First order benefits** will usually take the form of simple measures of goal achievement such as:

- millions of gallons per day of drinking water,
- megawatts of installed electrical generating capacity,
- electrical energy production over time,
- foregone property damage and loss of life avoided from improved flood control,
- increased domestic crop production as a result of improved irrigation; and

- efficiencies and other savings associated with improved river navigability.

In less developed nations, recreational benefits are, more often than not, viewed as an unaffordable luxury. Nonetheless, it might be possible to identify real and measurable (though unplanned) economic benefits of new impoundments and controlled water flows which may result from recreational exploitation of dam projects. Likewise, although a new impoundment will have a substantial negative effect on river ecosystems, at the peripheries of the impoundment this effect is likely to be positive.

First order costs take the form of the obvious impacts on the physical and human environment. They include:

- the numbers of people in the newly impounded area who will be displaced, and the monetary costs of resettlement and rehabilitation;
- the loss to inundation of land previously used and useful for forestry or agricultural cultivation;
- the immediate environmental impacts upstream and downstream of the proposed project, including increased sedimentation and siltation in the impoundment, loss of existing fisheries and other aquatic and wildlife habitat; upstream and downstream bank erosion; modification of water quality; loss of or reduction in particular animal and plant species; possible waterlogging and salination of surrounding lands; possible increases in seismological risk; and possible increases in waterborne diseases.

The measurement of these costs tends to be largely a technical exercise. However, this does not mean that these determinations are simple. As may be seen from the attempt to develop predictions about reservoir siltation, this can be an extremely complicated exercise. The rate of siltation in any particular reservoir may be related to numerous variables including climate, plant cover, rainfall, river hydrology, geology, storage capacity, topography, reservoir length and shape, and present and future land uses in the river basin.

SECOND ORDER BENEFITS AND COSTS

Unquestionably, the analysis of first order costs and benefits is an extremely complex undertaking -- certainly more than can realistically be accomplished by even a sophisticated analytical organization within the 120 day window imposed by the Pelosi amendment. If to this is added consideration of the second order costs and benefits of hydroelectric project lending decisions, the analytical task facing the lending officer becomes very daunting indeed.

Second order benefits include:

- economic development and modernization of underdeveloped nations,
- improved quality of life resulting from electrification,
- improvement in public well being resulting from stable agricultural productivity due to irrigation,
- improved public health resulting from improved drinking water quality.

In addition, improvements in agricultural productivity can free up more of the population for other commercial activities and provide, together with improved water supply and electrification, a *bona fide* entrance to first world status.

The **second order costs** associated with hydroelectric projects can also be significant, though extremely difficult to measure. Fundamental changes in culture(s), income distribution, and resultant changes in political power can in turn have a major effect on future decision making in the nation.

For example, in recent months there has been great controversy about the impact of the proposed expansion of James Bay Hydro in Quebec. Critics argue that the changed aquatic habitat of the massive project reservoir will not only modify the fisheries composition of the river basin, but will fundamentally and irrevocably alter the way of life of the Cree and Innuit indian tribes who have, for centuries, lived off the river's bounty. Project advocates argue that the burden of this change on the indian tribes is being handsomely rewarded, while critics seem more concerned about the loss of another traditional culture in an increasingly uniform world. Similar criticisms have been made of the Sardar Sarovar project in India, where

fundamental alterations of indigenous fish, wildlife and vegetation along the Narmada River will irrevocably destroy the natural basis for existing tribal cultures.

Given the broad social change that can result from large scale dam projects, it is also tempting to view project benefits in terms of raw political power and income distribution. Undeniably, the cultural, economic and social upheaval caused by population relocation and rehabilitation, is a cost which is borne by only a segment of the population. While there may be compensatory benefits to the same affected populations, it is possible to identify numerous different groups which benefit more or less from any particular project. The poor, the middle class, the rich, the city dweller, the rural peasant, the farmer -- all will be affected differently.

Consideration of these second order effects brings us full circle to a discussion of the broad social and economic goals which may have motivated the project in the first place. For the lender, analysis of these effects may be an interesting exercise, but they should, rightfully, be outside of his/her domain. They can not be evaluated as a mere technical exercise and are best left to be evaluated in the political arena. While this conclusion may not sit well with those who seek political justice for the world, we note that our conclusion is not an abdication of this goal. Rather, it simply suggests that this goal is better achieved in other arenas than the boardrooms of lending institutions.

U.S. Army Corps of Engineers
Major Rehabilitation Program, An Overview

Craig L. Chapman, PE[1]

Abstract

The U.S. Army Corps of Engineers (Corps) has established a process for evaluating the rehabilitation needs of its water resources projects. This paper is an overview of the Corps' water resources activities and the major rehabilitation evaluation program. Three associated papers, *Evaluating Power Benefits for Powerhouse Rehabilitation Studies* (Mittelstadt, 1993); *Reliability Analysis of Hydropower Equipment* (Norlin, 1993) and *Use of Event Trees and Economic Models in Evaluating Hydropower Rehabilitation Projects* (Obradovich, 1993) detail the risk analysis of hydropower equipment, power benefits and the economic models that go into the evaluation process.

Introduction

This paper gives an overview of the Corps' major rehabilitation program. This program encompasses hydropower, flood control and navigation facilities operated by the Corps. The general evaluation policies and processes discussed here are applicable to all three functional areas. The evaluation process for hydropower will be the focal point. Starting with a brief review of the operation and maintenance (O&M) program, the discussion leads to the need to establish a methodology for setting priorities for rehabilitation and efficiency improvements.

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[1]Hydropower Coordinator, Operations Branch, Headquarters, U.S. Army Corps of Engineers, 20 Massachusetts Ave., NW, Washington, DC 20314-1000

The Corps operates and maintains 75 hydropower projects across the country. This 21,000+ mW capacity represents approximately one quarter of the U.S. hydropower capacity and will average nearly 40 years of age by decade's end. Although age alone is not sufficient justification to replace capital equipment, the older the plant, the greater the "opportunity."

Risk analysis and its application to economic evaluation techniques is the key to developing this methodology. The papers cited earlier detail these techniques. The concept of risk analysis, event trees and power benefits are among the topics discussed.

What is the Corps Doing in Water Resources?

The Corps' beginnings are as old as this Nation's. Its original mission was that of the military engineer. Corps combat engineers have served in every conflict since the Revolutionary War. In the 1820s, Congress gave the Corps a mission to clear snags on the Ohio and Mississippi Rivers, to help this country's westward expansion. By the beginning of the 20th century, the Corps had established its role in water resources. In the 1930s, the Corps began building two large multi-purpose projects: Bonneville Dam on the Columbia River, and Fort Peck on the Missouri River. The mission of the Corps is ever changing. During World War II, the Corps built every type of military facility imaginable. At the height of the Cold War, the Corps was building air bases and missile silos. Today, its direction is changing. While the Corps still provides support to the Army and Air Force, a mission that has become increasingly important is environmental restoration of current and former defense installations.

The Corps operates and maintains more than 1,500 water resources projects. The primary purposes for which Corps projects are operated are one or more of the following:

Flood Control
Hydropower
Navigation
Recreation and Natural Resources

Other project purposes include irrigation, municipal/industrial water supply, water quality, and other related purposes such as sediment control and saltwater intrusion.

The Corps has designed and constructed water resources projects in virtually every state in this country. Most all the navigation, recreation and hydropower facilities are operated and maintained by the Corps. Many smaller flood control projects are designed and constructed by the Corps and transferred to local entities to operate and maintain.

All projects begin with Congressional authorizations and appropriations. A project is authorized either by specific legislation, or under general authorizations such as the Water Resources Development Act of 1992 (WRDA 92). WRDA 86 requires cost sharing for planning, design and construction of new projects and major rehabilitation of existing facilities. The level of cost sharing is dependent on the type of project. The funding of the rehabilitation of inland navigation projects is shared 50 percent with the Inland Waterways Trust Fund. However, new hydropower development is funded 100 percent by a non-Federal sponsor. Routine hydropower O&M and major rehabilitation continues to be a Federal responsibility.

The costs of Federal hydropower construction and O&M is recovered by the four Federal Power Marketing Administrations (PMAs) that market the Corps' generation. The PMAs set the power rates that they charge their customers at a level that covers their costs plus repayment of the Corps capital and annual O&M expenses to the U.S. Treasury.

Corps Hydropower

The Corps owns, operates and maintains 75 hydropower projects with a total nameplate capacity of 20,719 MegaWatts (MW), which represents approximately one quarter of all hydropower capacity in this country (See Figure 1.). This makes the Corps the fourth largest electric capacity owner. The Tennessee Valley Authority (TVA) is the largest, approximately 50 percent larger than the Corps; however, the majority of their capacity is fossil and nuclear. The Federal presence in the U.S. electrical industry is significant when you consider that three of the 11 largest capacity owners are affiliated with the Government: TVA (1), Corps (4) and U.S. Bureau of Reclamation (11). The Corps' 20,000+ MW of capacity produces an average of 70 billion kW-Hrs of energy each year. From the sale of this energy, the PMAs return \$400 to \$500 million each year to the U.S. Treasury for capital recovery and current year routine O&M.

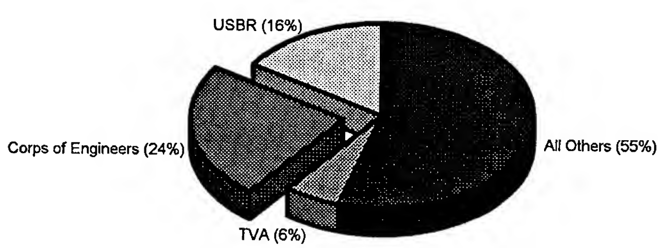

Figure 1. - Total U.S. Hydropower - 87,000 mW

Because of the multi-purpose nature of Corps projects, operational decisions at times run counter to the optimization of power production. In the upper Missouri basin, releases must be controlled to maintain constant stages below the projects during most of the summer to protect the nesting habitat of least tern and piping plover. The reduction in capacity during this time is more than 200 MW. On the Columbia and Snake Rivers, the listing of several species of salmon as either threatened or endangered will have an impact much, greater than that of the nesting birds. The Corps is proud of it's ability to balance all of these competing purposes.

The Corps has 349 main generating units that have an average age of nearly 30 years. The oldest, Bonneville Unit 1 is now 54 years old. The service lives of most major electrical and mechanical equipment in the powerhouse is within this range. In addition to these turbines and generators, all of the powerhouse auxiliary systems, main transformers, switchyards and other miscellaneous project facilities are aging. While equipment is not replaced based on age, at some point in time normal maintenance and repairs rise to uncomfortable levels. The issue then becomes: how much money and when?

Funding: History and Trends

There are three Congressional appropriations that fund the majority of Civil Works activities: General Investigations (GI), Construction, General (CG) and Operations and Maintenance, General (O&M (Gen)). The normal funding source for an operational Corps hydropower facility is through the O&M (Gen) appropriation. As seen in Figure 2a., funding for O&M through Fiscal Year 1990 (FY90) was nearly flat. The operations dollars shown are primarily labor and small service contracts such as janitorial services at the powerhouse. The maintenance dollars shown are the totals for

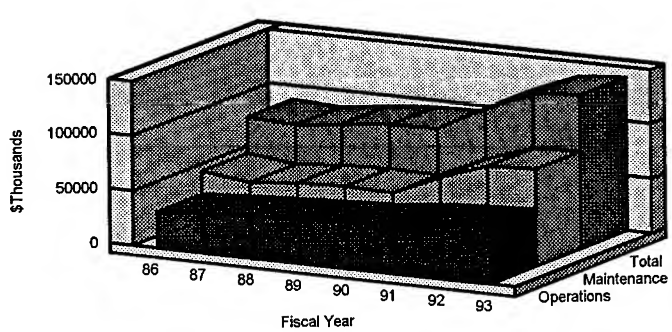

Figure 2a. - Hydropower O&M Funding
- FY86-90 Actual Expenditures, FY91-93 President's Budget

both recurring (routine) maintenance and non-recurring (non-routine) maintenance, improvements and betterments. It should be noted that the non-recurring costs may be either expenses or capital investments, while the recurring costs are almost always an expense.

As can be seen in Figure 2b., non-recurring funding dropped off considerably in FY87-90. This was the result of the continuous pressure to hold the line on Federal budgets. For FY91 and after, headquarters operations management made a decision to increase the funding for non-recurring hydropower maintenance. The increase was an attempt to reduce the backlog of maintenance items. This increase was done without additional appropriations, but with a change of allocation within our budget ceiling. In other words, other functions were impacted to accommodate hydropower.

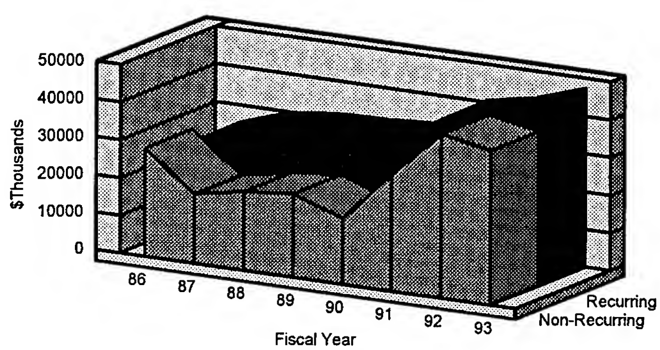

Figure 2b. - Hydropower Maintenence Funding
- FY86-90 Actual Expenditures, FY91-93 President's Budget

Need for the Major Rehabilitation Methodology

In the mid-1970s the Corps initiated a major rehabilitation program to provide a way to manage the large expenditures required to extend the life of its aging projects. Over the years the program and procedures have undergone many changes. The Water Resources Development Act of 1986 established the Inland Waterways Trust Fund and cost sharing of inland navigation rehabilitation. Better evaluation reports were needed.

In 1990, representatives from the Assistant Secretary of the Army (Civil Works) (ASA(CW)), Office of Management and Budget (OMB) and the Corps began working on improving the rehabilitation evaluation report guidance. The concept of risk analysis was added to the evaluation report. Since total replacement is not always the correct answer, all practical alternatives are investigated, including not doing anything other than continued maintenance and repair.

In addition to the engineering solutions, each alternative's environmental issues, economics and costs are evaluated. Refinements in the program continue as the Corps improves its risk analysis techniques.

Power Benefit Analysis

When evaluating any major hydropower rehabilitation plan, it is necessary to determine if the gain in benefits realized from the plan exceed the costs of implementing the plan. The benefits from rehabilitation measures fall into three broad categories:

◆ a gain in efficiency, such as would be realized by rebuilding or replacing a worn turbine runner;

◆ a gain in capacity, such as would be realized by rewinding a generator with state-of-the-art material; and

◆ a gain in reliability, such as would be realized by replacement of old, failure-prone components with new components.

The procedures for evaluating benefits attributable to gains in efficiency and capacity are well-established. However, determining the benefits of reliability improvements has required plowing some new ground. Data had to be developed on probabilities of failure versus age or condition for the major components of the power train, and an event tree model developed to translate this data into a gain in power benefits that would be realized from reliability improvement.

Reliability (Risk) Analysis

Probabilities of unsatisfactory operation (or failure) are developed for the project or a piece of equipment. Structures such as navigation locks use a methodology that looks at design capabilities, existing capabilities and expected loads to determine the probability of unsatisfactory operation. For electrical and mechanical equipment, especially in hydropower, the design life and Condition Index are used to develop the probabilities. For many items we use survivor curves that give the expected percentage of equipment that will be in service after a period of years. These are similar to the mortality tables that the life insurance companies use to set policy rates. The expected failure rate at any given time is a function of the Condition Index and the slope of the survivor curve. The better the condition, the lower the risk; the poorer condition, the higher the risk. By using simulation techniques, costs over time can be developed to complete the economic analysis.

Event Tree Models

An event tree is a diagram of the potential events and outcomes that could occur to a given component or group of components in one time period or in a series of time periods. The model developed by the Corps for analysis of reliability aspects of rehabilitation plans uses an event tree to determine the expected loss of energy benefits due to units being out of service. It estimates these costs over the service life of the proposed rehabilitation measures and also accounts for the additional O&M costs required to bring failed units back into service. Key input variables are the failure probability data developed from the risk analysis and the unit energy values developed in the earlier phases of economic analysis process.

Related Papers

In the remainder of this session, papers will be presented which address in more detail the last three topics discussed above. Dick Mittelstadt will review the overall approach to hydropower benefits analysis as applied to rehabilitation projects. Jim Norlin will describe the work the Corps has been doing in the area of reliability analysis, and Pat Obradovich will discuss the event tree models.

Summary

What's left to be done? The evaluation framework is developed to a point that it will be the way business will be done for the foreseeable future. Risk and reliability analysis will continue to be refined and expanded to cover more equipment and facilities. The processes that the Corps uses for hydropower work very well for generator stator windings and any other equipment that has good survivor curves. Improvements are needed in the evaluation procedures for governors, excitation systems and other peripheral equipment. Typically, this type of equipment and systems exhibit problems with poor performance instead of poor reliability.

Using the evaluation processes outlined, two hydropower major rehabilitation projects have been approved: the Bonneville switchyard and transformers and the Dardanelle turbines and generators, all FY93 starts. A third project, the rehabilitation of Bonneville Plant 1 turbines and generators has been recommended for the FY94 program. This hydropower work is in addition to several navigation rehabilitation projects for FY93 and beyond.

Acknowledgments

A special thanks goes to the authors of the associated papers and the staffs of the Corps of Engineers, Institute for Water Resources, Ft. Belvoir, VA and the Hydroelectric Design Center, Portland, OR.

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Reliability Analysis of Hydropower Equipment

By James A. Norlin, P.E.[1]

Abstract

Major rehabilitation of U.S. Army Corps of Engineers (Corps) hydropower projects is funded by specific Congressional appropriations. Projects compete against each other because funding is limited. An evaluation report is prepared to assure that the recommended rehabilitation is justified and maximizes net benefits. The report must include a rigorous technical and economic analysis and reporting. This process includes an engineering analysis that estimates the probabilities of unreliable service in future time frames. This paper describes the approach that the Corps is developing for determining the reliability of hydropower equipment.

Introduction

The Corps in conjunction with the Office of Management and Budget (OMB) has recently established a Major Rehabilitation Program for existing hydropower, navigation, flood control and coastal projects. Hydropower rehabilitation projects must be in excess of $5 million to qualify for this program. A major rehabilitation report must be approved by both the Corps and OMB before it can be submitted to Congress as part of the President's budget. The economic analysis used in these reports requires an engineering reliability analysis. The Corps has been working over the past two years to develop a method of analysis that is acceptable to both engineers and economists.

Definitions

The three following definitions are from Frank Knight in the pioneering book, *Risk, Uncertainty and Profit*, originally published in 1921.

[1] Mechanical Engineer, Hydropower Rehabilitation Specialist, Hydroelectric Design Center, Corps of Engineers, PO Box 2870, Portland, OR 97208-2870, (503) 326-6589

Certainty - A situation where the decision maker knows each possible alternative available and its exact outcome.

Risk - A situation where the decision maker knows all of the alternatives available but each alternative has a number of possible outcomes. Thus, the decision maker no longer knows the outcome of each alternative. In this region, probabilities are assigned to each outcome. This is further broken down into two types of risk, objective and subjective. Objective risk means that the probabilities are objectively estimated. One of the best examples of this is historic stream flow data. Subjective risk relies on peoples beliefs about the likelihood of events.

Uncertainty - A situation where probabilities cannot be assigned to the outcomes. Only some of the alternatives or their outcomes may be known. In the extreme, decision makers may be faced with complete ignorance.

All three of these situations are represented when discussing the reliability of hydroelectric powerplants. A generating unit that has been derated because of previous problems is not capable of reliably producing the same amount of power that it could originally produce. That is a certainty. How long that unit can keep producing any power at all is a situation involving risk. It may involve objective risk if there is enough historical data available to project the probability of a future failure of the equipment. An example of uncertainty in a hydropower rehabilitation project may involve the highly unlikely possibility of a catastrophic failure of a turbine component that would result in a total flooding of the powerhouse.

The Corps defines hydropower equipment reliability as:

> The extent to which the generating equipment can be counted on to perform as originally intended. This encompasses 1) the confidence in soundness or integrity of the equipment based on maintenance costs and forced outage experience, 2) the output of the equipment in terms of measured energy, power, efficiency, and availability, and 3) the dependability of the equipment in terms of remaining service life (retirement of the equipment).

The engineering reliability analysis performed by the Corps looks at each applicable component of reliability separately. The following paragraphs briefly summarize each phase of the analysis.

Maintenance Costs and Forced Outage Experience

The total cost of equipment maintenance and repair includes labor and materials costs as well as lost energy and capacity benefits. The rate of change of the labor and materials costs (over and above inflation) can be used as an indicator of reliability. Lost

energy and capacity are discussed under the topic of availability. Increasing maintenance costs and unit outage hours can both be used for justifying equipment replacement or rehabilitation. Project records for the equipment item in question can be used to establish past trends and future projections. This is usually the sole justification that is required for replacing relatively low cost items that are critical for power production. Increasing maintenance and forced outages are the key element in placing work into the major rehabilitation program. Projections of future maintenance costs and forced outages are handled as subjective risk if adequate supporting information is available.

Efficiency and Capacity

This portion of the reliability analysis can be applied to any piece of equipment that has an affect on the ability of the generating unit to produce rated power at rated efficiency, and/or on the unit's availability. This approach is applicable primarily to the turbines, generators and transformers. Turbines will be used as an example in the following explanation.

Part of the aging process of turbines is the development of cracks, corrosion, erosion, scaling and cavitation damage. Much of this damage is corrected by welding, which can change the shape of the turbine water passage. Thus, the performance of the turbine deteriorates as a result of both the aging process and the repair.

The first step in quantifying this deterioration is to determine current levels of performance. Power output and efficiency must be determined by performance testing. The current performance must then be compared with the original level of performance to establish the amount of performance degradation that has occurred. Original levels of performance can be established from model test or acceptance test data.

The quantified loss in efficiency and/or capacity is handled in the risk analysis as a certainty. Continuing degradation can either be ignored or handled as objective or subjective risk depending upon the information available.

The negative effects of a degraded generating unit can be directly

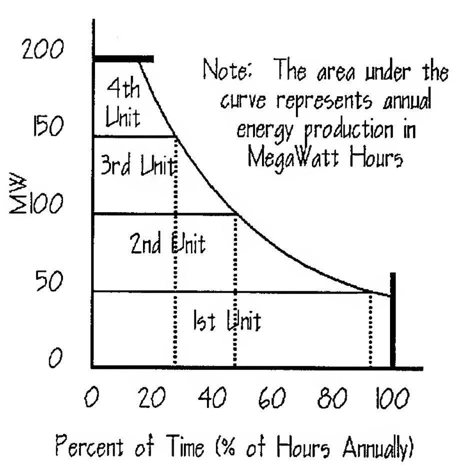

Figure 1. - Generation Duration Curve

calculated by determining the quantity of energy (megaWatt hours) that is lost and multiplying by the value of that energy (dollars per MW hour). The amount of energy that is lost should be quantified through the use of a generation duration curve. Figure 1 shows a typical generation duration curve for a powerhouse with 4 - 50 MW units.

This curve is unique for each project, and is based on the size, number and efficiency of the units, and the annual water available for power generation. Figure 1 shows that one of the 50 MW generating units is required at all times. Approximately 95 percent of the time, a second unit is required to capture the available energy. The third unit is required approximately 45 percent of the time and the fourth unit only 25 percent of the time. Thus, in this hypothetical instance, a single unit on outage would only cause a loss in the capability to generate the available energy 25 percent of the time. However, the cost of a single unit on outage may be quite high depending upon the regional value of energy and capacity.

Availability

Availability is the annual percentage of time that generating equipment is available for power production. Records of availability are maintained by each project on a unit by unit basis. The current level of availability must be compared with previous data to establish the extent of degradation. Historical trends can be extrapolated to project future changes in the unit availability rate. This is considered as objective risk if there is enough supporting information.

Retirement

The final area of consideration concerning equipment reliability is retirement, or projecting the remaining service life of the equipment. The Corps defines retirement as the removal of a piece of equipment from service for any reason. Two major reasons for retirement of a piece of equipment are; 1) physical condition, which includes deterioration from time, wear and tear from use and catastrophe; and 2) functional situations, which includes inadequacy to perform required functions, potential for improvement (uprating) and obsolescence.

The first category, physical condition, is the primary reason that the Corps developed the Major Rehabilitation Program. This program establishes a standardized method of considering and evaluating the deterioration and wear of equipment in an effort to prioritize repairs and replacements based on need as well as value. Catastrophes are generally handled through reprogramming O&M funds.
Improvements in functional situations at projects that have minimal deterioration are considered by the Corps as project modernization and generally have a lower priority than reliability improvements.

Predictions of remaining life, or determining the probability that the unit will fail to perform satisfactorily has historically been done using pure engineering judgement (subjective risk). While most engineers are comfortable with this approach, most economists are not. Thus, the Corps has embarked on a program to attempt to quantify these probabilities so that the risk is analyzed in both an objective and subjective manner.

Methods of determining probable remaining service life are well established for many types of physical properties. However, the probable remaining service life is not suitable for the economic analysis required by the Corps. The economic analysis needs data in terms of the annual probability that a piece of equipment will need to be retired (replaced or rebuilt). The established methods of determining remaining service life can be adapted to meet our needs by obtaining the frequency of retirement values from survivor curves.

The statistical approach has been used by insurance companies for centuries to predict human mortality. The methodology was adapted early in the 20th century to predict the service life of physical properties to determine depreciation rates.

The Corps is assembling a large database of equipment histories to estimate the probability of the need to retire a specific component in any given future time period. The historical data includes the year installed and the year of retirement. The initial work in this area focuses on generator stator windings because there have been a significant number of stator retirements in the form of rewinds. The baseline retirement statistics for generator stator windings was originally developed by using historical retirement information for the Corps. This database has been supplemented by adding information from several large hydropower producers in North America.

Age	Year Installed	Number Installed	Number Still in Service	Percent Surviving
31	1962	9	9	100.00%
32	1961	9	5	55.56%
33	1960	18	15	83.33%
34	1959	35	27	77.14%
35	1958	28	19	67.86%
36	1957	18	9	50.00%

Table 1. - Summary Survival Data

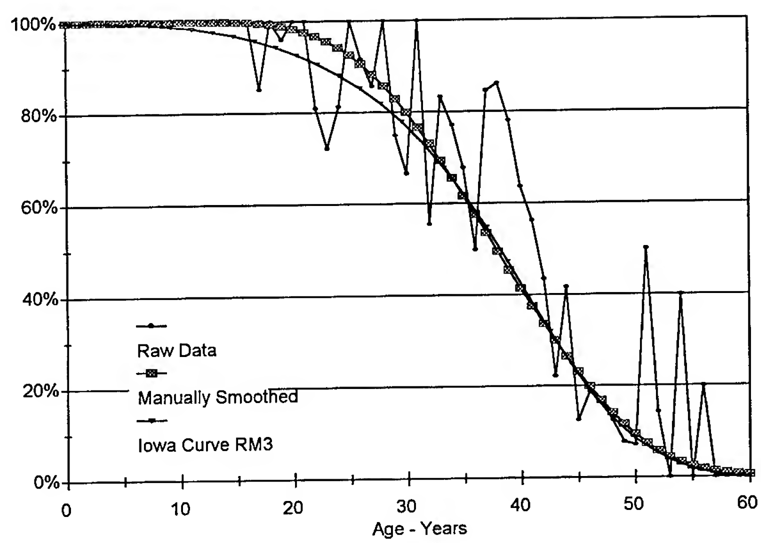

Figure 2. - Typical Survivor Curve

The raw data is compiled and reduced into annual summaries. Table 1 shows a small portion of the annual summary data. The data from Table 1 is plotted with Age as the abscissa and Percent Survivors as the ordinate. The resulting plot is shown in Figure 2 as the jagged line.

The raw retirement data can be smoothed using any number of means. The most widely known and universally accepted method is the application of "Iowa Curves" developed in the 1930s by the Engineering Experiment Station at what was then Iowa State College (Winfrey, 1935). This work sets forth the principles for probable life, average life, life expectancy, etc. of property surviving in service for any given age of service. The studies looked at many different types of industrial property and developed the mathematical equations for 18 typical survivor curves (Type Curves). Four additional Type Curves were added to the 1967 edition of Bulletin 125, bringing the total number to 22.

Selecting the appropriate Iowa Curve to fit the data shown in Table 1 and represented in Figure 2 is typically accomplished by plotting both curves to the same scale and overlaying them until the best observed fit is realized. The method recommended by Winfrey is to manually smooth the survivor curve by plotting and smoothing out annual differences. This is relatively easily accomplished using spreadsheet software. The resulting curve should be smooth flowing and regular in

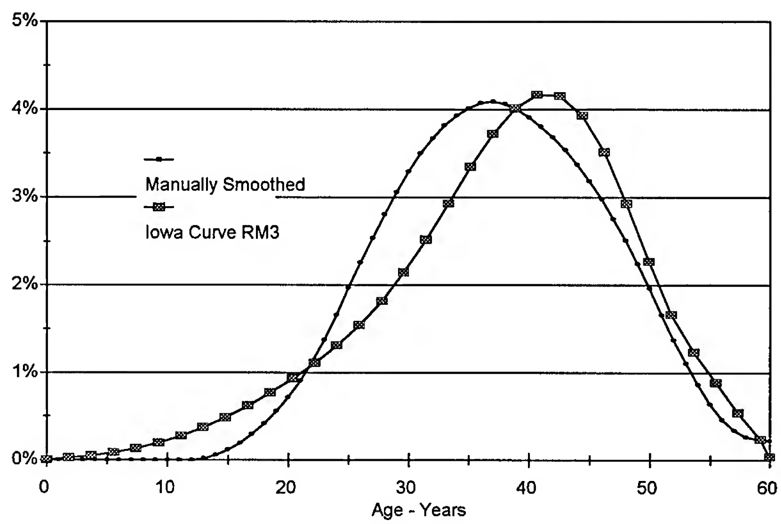

Figure 3. - Typical Frequency Curve

curvature. The area under the smooth curve and the original data curve should balance closely. The manually smoothed curve is also represented in Figure 2.

Plotting smoothed frequency curves can assist in making the appropriate adjustments. An example of the frequency curve is shown in Figure 3. The frequency curve is the slope of the survivor curve at any given age.

The next step in the process is to overlay the Iowa Type Curves to obtain as close a match as possible. This is also represented in Figures 2 and 3. The curve that was chosen to represent the preliminary database for generator stator windings is the Right Modal Curve Number 3 (RM-3). This curve matches the lower portion of the manually smoothed survivor curve and the manually smoothed frequency curve very closely. The upper end does not match quite as well. The amount of raw data at the lower end of the curve is relatively small. As additional data is collected and added to the database, it is anticipated that there will be a greater percentage of retired units in the 50 to 60 year age group that is shown. If this proves to be true, then the RM-4 curve may prove to be a better fit than the RM-3 curve.

The final step in the process is to plot the probable remaining life curve along with the survivor curve. This is represented in Figure 4. In this example, the stator winding being evaluated is 33 years old. The probability of failure is determined by the triangular distribution of probabilities as shown. These data are used by the

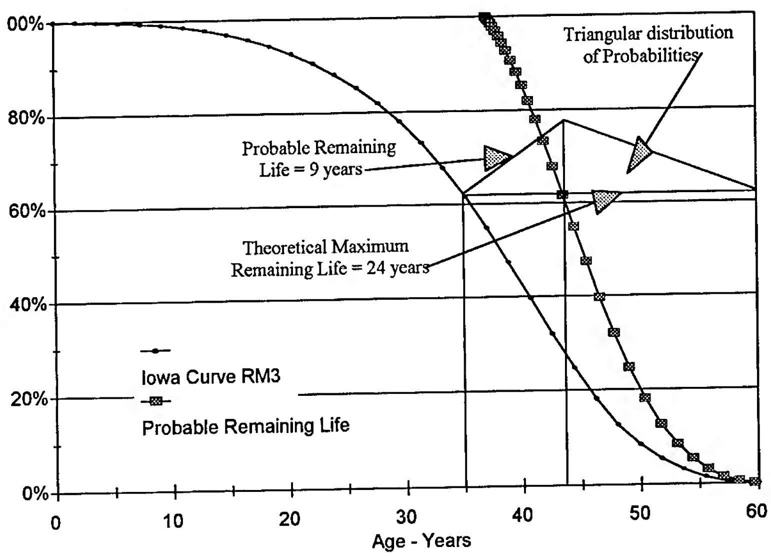

Figure 4. - Probable Remaining Life Curve

economists in a Monte Carlo simulation to evaluate the probabilities of potential future retirements.

If the specific equipment in question can be considered to be "average" in all respects, then this frequency data can be applied directly. However, if the equipment is exhibiting signs of premature or accelerated deterioration, appropriate modifiers may be applied to the generic curve in order to account for the higher probability of early retirement. Likewise, modifiers may be applied to account for lower probabilities of retirement for equipment that is in very good condition.

The factor used by the Corps to modify the frequency curve data is the Condition Indicator (CI). Condition Indicators have been developed by the Corps for many types of equipment and structures. They are first and foremost a screening tool, providing a uniform method of evaluating condition through testing and inspections. Inspection and test data are gathered for each unit in accordance with the latest Condition Indicator Manual. Condition Index numbers are then assigned based on this information and the CI Manual. Table 2 shows the Condition Index Scale.

Equipment with CI values from 70 to 100 is considered to be in excellent or very good condition. CI values in this range, when applied to the survivor curve, will tend

to decrease the probability of retirement. However, equipment in good condition is not a candidate for major rehabilitation and a detailed analysis is not generally required. Equipment with CI values in the mid-range, from 40 to 69, is considered fair to good. There is really no good reason to adjust the baseline frequency curve for equipment that falls into this category. Equipment with a Condition Index below 40 is considered to be in poor condition or worse. CI values below 40 will tend to increase the probability of retirement. The Corps is still working to determine a reasonable and rational method of applying the CI to the probability of retirement.

Future Work

- The Corps plans to increase the size of its databases for generator stator windings and turbine runners. Participants in this effort so far include the Bureau of Reclamation, the Tennessee Valley Authority, Southern Company Services, Ontario Hydro and BC Hydro.

- The Corps is considering performing an analysis of specific subgroups of equipment. An example of this would include stator windings by type of insulation.

Value	Condition Description	
85-100	Excellent	No noticeable defects. Some aging or wear may be noticeable.
70-84	Very Good	Only minor deterioration or defects are evident.
55-69	Good	Some deterioration or defects are evident, but function is not significantly affected.
40-54	Fair	Moderate deterioration. Function is still adequate.
25-39	Poor	Serious deterioration in at least some portions of equipment. Function is inadequate.
10-24	Very Poor	Extensive deterioration. Barely functional.
0-9	Failed	No longer functions. General failure or failure of a major component.

Table 2. - Condition Index Scale

- The Corps is planning to locate or develop databases for other classes of hydropower equipment is also planned. Initially this would include transformers and circuit breakers. Governors and excitation systems may also lend themselves to an analysis of this nature.

- Finally, the Corps plans to continue to refine the application of Condition Indicators as modifiers to the probability of failure.

Summary

The reliability analysis that the Corps is performing on hydropower equipment that is being considered for major rehabilitation consists of several components. These include an objective analysis of increasing maintenance costs, forced outages and availability. It also includes a quantitative analysis of lost performance in terms of efficiency and capacity. Finally, it includes an probabilistic analysis of when the equipment might need to be retired.

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Evaluating Power Benefits for Powerhouse Rehabilitation Studies

Richard L. Mittelstadt[1]

Abstract

Many U.S. Army Corps of Engineers (Corps) hydropower projects have been in operation for 30 years or more and some of these projects now require major rehabilitation (rehab) work. However, before funds can be allocated for such work, an analysis must be performed to determine if the work is economically justified, and if it is justified, what combination of rehab measures is optimal. A key part of the economic analysis is development of the benefits that will be realized from the rehab work. This paper describes how the Corps estimates those benefits.

Introduction

As Chapman indicated in the first paper in this session, the tight Federal budget has required that rehab projects be subjected to a much more rigorous economic analysis than in the past. Not only is it necessary to show that the dollar benefits of major rehab work exceed the cost, but it must also be demonstrated that each component in a rehab plan is incrementally justified and that that combination of components is the plan that yields the maximum net benefits. In short, proposals for major rehab work must be supported by the same level of economic analysis as new water resource development projects.

The purpose of this paper is to briefly review how the Corps of Engineers applies the principles of hydropower benefit analysis to determining the feasibility of powerhouse rehab projects. Emphasis will be placed on describing the different types of power benefits that can accrue to rehab projects. One of the more interesting aspects is the impact of risk and reliability on project output and project

[1] Hydropower Engineer, North Pacific Division, Corps of Engineers, P.O. Box 2870, Portland OR 97208-2870.

benefits. This subject will be touched on briefly in this paper but will be discussed in some detail in the following papers by Norlin and Obradovich.

Hydropower Benefit Analysis

Traditionally, the economic feasibility of a hydroelectric project is determined by comparing of the cost of the hydro project to the cost of the most likely thermal alternative. In other words, is the cost of constructing and operating a hydro project less than the cost of obtaining the power from the thermal powerplant(s) that would be the most likely source of that power if the hydro plant were not built?

Energy vs. Capacity Benefits

Two parameters define the output of a hydroelectric project: energy (the total amount of generation in a given period, expressed in megawatt-hours), and capacity (the maximum amount of power that can be delivered at any given time, expressed in megawatts).

Capacity benefits are measured as the cost of constructing an equivalent amount of thermal powerplant capacity. The capacity benefits represent the capital costs and other fixed costs associated with the most likely thermal alternative.

Energy benefits are measured by the cost of producing an equivalent amount of generation in the power system with the hydro plant replaced by the most likely thermal alternative. The energy benefits represent the variable costs associated with producing the alternative thermal generation, which are primarily fuel costs.

Gain in Output Resulting from Rehab Projects

The first step in estimating the benefits is to determine the gain in power output that will be realized from the proposed rehab plan.

Rehab measures can be grouped into three categories, based on the way in which they increase hydropower project output:

- those which increase efficiency
- those which increase capacity
- those which increase reliability

Replacing worn turbine runners would be an example of a measure that increases efficiency. Rewinding generators with state-of-the-art materials often permits the units to operate at higher output levels, and this would be an example of a capacity-increasing measure. Replacing aged stator coils with new coils in order to reduce coil failure outages would be an example of a measure which increases reliability.

Example

The easiest way to describe the benefit evaluation process is to walk through an example of a typical rehab project. Assume the proposed plan includes replacing all four worn turbine runners with new runners and rewinding the generator stators.

It will be assumed that when the original runners were new, the units had an average overall efficiency of 87 percent and that tests have shown that, in their current condition, the overall efficiency has dropped to 84 percent. With new runners, it is estimated that an average efficiency of 89 percent could be achieved. However, the rated capacity of the turbines remain the same.

The rated capacity of the original generators was 50 MW. By rewinding the generator stator with state-of-the art materials, the rated capacity of the generators can be increased to 60 MW, which now matches more closely the maximum capability of the turbines.

Duration Curve

To graphically display the amount of energy that could be gained from a rehab measure, a generation-duration curve will be used. The curve could be developed using historical records or output from a sequential streamflow routing model such as HEC-5.

Figure 1 shows the annual generation-duration curve for the example plant for the available period of streamflow record based on the existing condition of the plant. The table below shows the output of the plant by unit.

unit	--cap (MW)--		-energy (MWh)-	
	unit	cum	unit	cum
1	50	50	412,000	412,000
2	50	100	254,000	666,000
3	50	150	112,000	778,000
4	50	200	23,000	801,000

cum = cumulative; cap = capacity

Table 1. Plant Output by Unit: Existing Condition

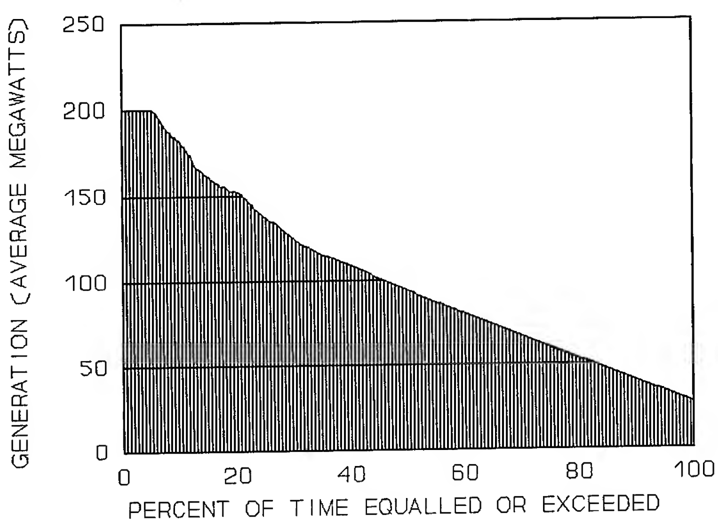

Figure 1. Generation-Duration Curve: Existing Condition

The duration curve in this case is based on weekly average streamflow data from a 60-year simulated operation study. It does not reflect the effect of peaking operation. This would require an hourly generation-duration curve, which would have the same area under the curve but would show more operation at or near full output and less operation at low output levels.

However, for purposes of estimating energy output, a curve based on average daily, weekly, or monthly output should be used rather than an hourly curve. The use of average values is necessary to measure the amount of energy that would otherwise be spilled if the rehab measure were not implemented.

The horizontal line at the top of the duration curve defines the maximum capacity of the plant, which in this case is 200 MW, the combined capacity of the four existing generators.

Energy Gained by New Runners

Figure 2 describes the gain in energy achieved by replacing the worn existing turbine runners with new state-of-the-art runners. The middle curve shows the output when the original runners were new (overall efficiency of 87 percent), and

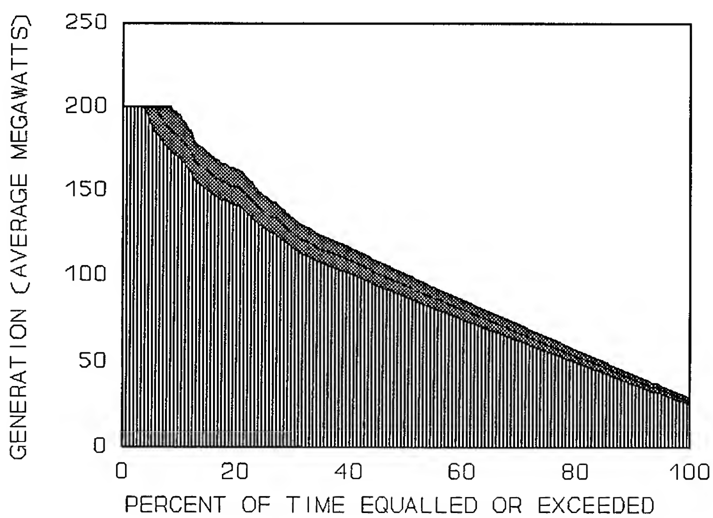

Figure 2. Generation-Duration Curve Showing Gain in Output With New Runners

the lower curve shows the output with the original runners in their existing, worn condition (overall efficiency of 84 percent). The upper curve shows the output with new state-of-the-art runners (overall efficiency of 89 percent). The dark shaded area represents the gain in energy creditable to the new runners. The upper and middle curves were derived by applying efficiency adjustment factors[2] to each of the points that was used to derive the existing case (Figure 1) generation-duration curve.

Energy output with original runners when new	828,000 MWh
Energy output with existing original runners	801,000 MWh
Energy output with new runners	845,000 MWh
Gain in energy output	44,000 MWh

Note that the capacity of the existing generators limits output to a maximum of 200 MW. So, even if the new runners had a somewhat greater megawatt capability, it would not be possible to take advantage of that capability.

[2] 0.87/0.84 for the original runner case and 0.89/0.84 for the new runner case.

Energy Gained by New Generator Windings

Figure 3 describes the gain in energy achieved by rewinding the stators with state-of-the art-insulation materials. The new materials make it possible to place more copper in the windings, which increases the capacity of the generators. In this example, it is assumed that the new runners are in place and the capacity of the generators can be increased to match the full output of the new runners. As a result, the capacity of the plant is increased to (4 x 60 MW) = 240 MW.

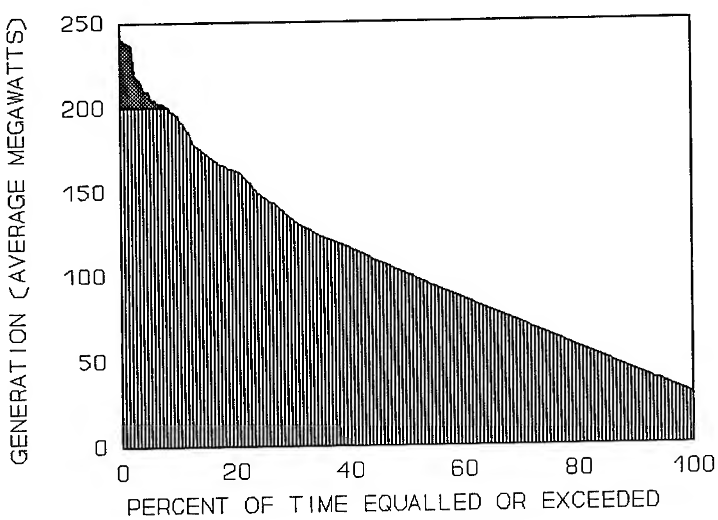

Figure 3. Generation-Duration Curve Showing Gain in Output with Rewound Generators

The upper limit (which truncates the duration curve) is increased from 200 MW to 240 MW, so the generation-duration curve was extended to the new upper limit. The dark shaded area on Figure 3 defines the gain in energy output realized from adding a generator rewind to turbine runner replacement.

Energy output with existing generators	845,000 MWh
Energy output with generator rewind	861,000 MWh
Gain in energy output	16,000 MWh

Note that a gain in generation could also be realized by rewinding the generators but retaining the existing turbines. The dark shaded area would be

smaller, being defined by an extension of the lower curve on Figure 2 rather than the upper curve. The gain in energy for this scenario would be 4,000 MWh instead of 16,000 MWh.

Energy Gained by Improved Reliability

Both the new runners and the generator rewind could contribute to improved reliability for the plant. Replacing old failure-prone components with new components usually reduces the amount of time generating units are out of service due to forced outages. This in turn increases the amount of generation the plant can produce.

Figure 4 illustrates the concept of generation loss due to forced outages. The dark shaded area represents the generation that would be lost if forced outages kept one unit out of service one-third of the time (high value assumed for illustrative purposes only, forced outage rates for hydro plants are typically less than ten percent). A rehab measure which reduces the outage rate would reduce the size of the dark shaded area, thus increasing energy output.

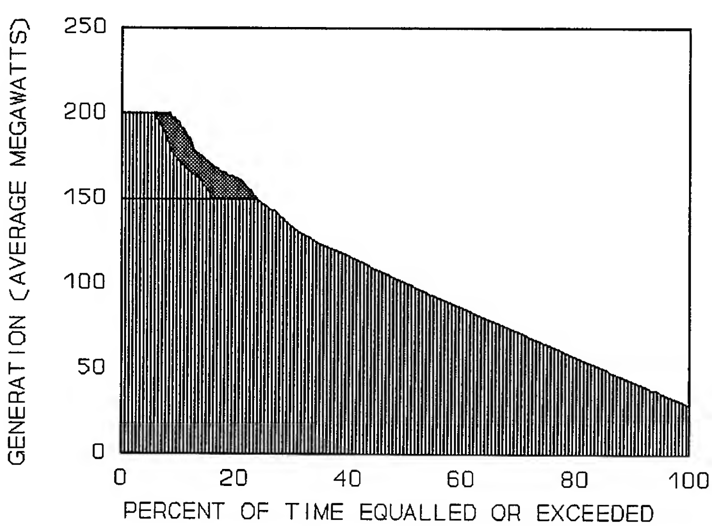

Figure 4. Generation-Duration Curve Showing Reduction in Output Due to Forced Outages

The process of computing the loss in energy due to outages is rather complex because it is necessary to account for (1) the combined probability characteristics of multiple components (turbine runners and generator windings, for example), (2) the combined probabilities of different numbers of units being out of service, and (3) the fact that component reliability tends to decrease with age. In addition, it is necessary to account for the length of the outage and the cost of repair. In order to account for all of these factors, event tree models have been developed for estimating the energy benefits attributable to reliability improvements. This topic is discussed in more detail in another paper in this session (Obradovich, 1993). However, for purposes of illustration, it is assumed that the combined gain in average annual energy benefits due to improvement in the reliability of the turbines and generators is $750,000.

Computation of Energy Benefits

The average annual gain in energy benefits that accrues to a rehab plan is computed by applying a unit energy value to the gain in energy creditable to that plan. Assuming an energy value of 30 mills/kWh, the gain in energy benefits for the runner replacement and generator rewind measures would be as follows:

Runner Replacement Benefits	=	(44,000 MWh x 30 mills/kWh)	=	$1,320,000
Generator Rewind Benefits	=	(16,000 MWh x 30 mills/kWh)	=	$480,000
Reliability Benefits			=	$750,000
Total Energy Benefits			=	$2,550,000

The unit energy values represent the energy cost associated with producing the generation with the most likely thermal alternative. The 30 mills/kWh energy value might be based, for example, on a mix of coal-fired steam and gas-fired combustion turbines. The Corps usually develops these values using a system production cost model, simulating the operation of the power system twice: once with the hydro project in the system and once with the hydro plant replaced with an equivalent number of megawatts of thermal capacity (Mittelstadt, 1991).

Dependable Capacity

The dependable capacity of a hydropower plant is an estimate of the amount of thermal generating capacity that would carry the same amount of peak load in a power system as the hydropower plant. It is intended to account for the variables that affect the amount of hydropower capacity that can be used effectively in the system load, including:

- the variability in the maximum capacity that a hydropower plant can deliver caused by variations in head

- the variability in usable capacity caused by variations in the availability of streamflow, which in turn cause variations in the amount of energy available to support the capacity

A variety of different techniques are used to estimate dependable capacity. The Corps presently uses the average availability method for projects which operate in thermal-based power systems and the critical month method for projects in hydro-based power systems (Mittelstadt, 1989).

For this example, the average availability method was used. Space does not permit a detailed discussion of the procedure, but, in brief, it involves computing the amount of capacity that can be supported with the available energy for each week in the peak demand months for each year in the hydrologic period of record. The average capacity that can be supported over that period defines the project's dependable capacity.

Supportable capacity is defined as the amount of capacity that can be supported for a specified number of hours per week. The number of hours required varies from project to project and from system to system, depending on the system resource mix and hourly load shape. A typical example might be 4 hours per day, five days per week (or 20 hours per week).

Some examples will illustrate this concept. Taking the 200 MW example project and using the 20 hours/week criteria, assume that in a particular month, sufficient streamflow is available to produce 5,000 MWh/week. Applying the 20-hour criteria, (5,000 MWh)/(20 hours/week) = 250 MW could theoretically be supported. However, the installed capacity of the plant is only 200 MW, so the supportable capacity for that month is limited to 200 MW. However, if the generators were rewound to 240 MW, the supportable capacity would increase to 240 MW. Assume that in another month, 3,000 MWh/week can be generated. In this month, only (3,000 MWh)/(20 hours/week) = 150 MW can be supported, either with or without the rewind.

Dependable Capacity Gained By New Runners

The amount of energy available in each week will be increased due to the higher runner efficiency. In some weeks, sufficient energy is already available to support the existing capacity. But in some of the lower flow weeks, this additional energy will permit more capacity to be supported. The average gain in capacity over all of the peak demand weeks in the period of record defines the gain in dependable capacity attributable to the new runners. Typically, this gain is relatively small for runner replacement, and for this example, the new runners increase the dependable capacity from 185 MW to 190 MW (compared to an installed capacity of 200 MW).

Dependable Capacity Gained By Generator Rewind

This generator rewind increases the maximum capacity of the plant. This in turn permits more capacity to be supported in those weeks where more energy is available than is needed to support the existing capacity. In the example case, if the generator capacity is increased by 40 MW, the dependable capacity increases from 190 MW to 226 MW (compared to the new installed capacity of 240 MW).

Computation of Capacity Benefits

The average annual gain in capacity benefits that accrues to a rehab plan is computed by applying a unit capacity value to the gain in dependable capacity creditable to that plan. Assuming a capacity value of $120/kW-year, the gain in capacity benefits for the runner replacement and rewind measures would be:

Runner Replacement Benefits	=	(5 MW x $120/kW-year)	=	$600,000
Generator Rewind Benefits	=	(36 MW x $120/kW-year)	=	$4,300,000
Total Capacity Benefits			=	$4,900,000

The unit capacity values represent the investment cost associated with delivering the replacement capacity with the most likely thermal alternative. The $120/kW-year capacity value is based on a mix of coal-fired steam and gas-fired combustion turbines. The Corps usually obtains these values from the Federal Energy Regulatory Commission (FERC), although they can be developed from data published by the Electric Power Research Institute (EPRI, 1986) and other sources.

Increase in Capacity Benefits Realized By Increased Reliability

Although improving the electrical-mechanical reliability of hydroelectric generating units clearly increases the peak load-carrying capability of the units, it has proven difficult to quantitatively estimate the benefits realized from this gain. However, Garver has developed a relationship of generating unit average availability to effective load-carrying capability (Garver, 1966).[3] Using his equation, effective load-carrying capabilities (ELCC's) can be developed for each unit size and each

[3] Load-carrying capability (L) $= C - \{M * \ln[(1-R) + (R * e^{C/M})]\}$

where:

L	=	effective load-carrying capability of unit (MW)
C	=	rated capacity of that unit (MW)
M	=	system characteristic (typically, 3% of total system capacity in MW)
R	=	unit's equivalent forced outage rate (%)
e	=	2.718.

forced outage rate. Ratios of effective load-carrying capability can then be applied to the unit capacity values to estimate the gain in capacity benefits that apply to the proposed rehab measure or plan.

Table 2 illustrates how this concept can be applied to adjusting the unit capacity values. These values, as developed by FERC, already include a factor which accounts for the average availability of a typical hydropower unit compared to a thermal generating unit (US Water Resources Council, 1981). For the example study, assume that the \$120/kW-year FERC capacity value is based on a typical hydro unit availability of 93 percent, and the availability of the units in their existing condition is 91 percent. Assume that the turbine runner replacement increases the availability to 93 percent, and adding the generator rewind increases it to 95 percent. These availability values would be obtained from reliability studies (Norlin, 1993).

Case	Availability (%)	ELCC (MW)	Ratio	Cap Val (\$/kW-yr)
FERC Value	93	46.4	1.000	120
Existing	91	45.3	0.976	117
New Runners	93	46.4	1.000	120
Rewind	95	47.4	1.022	123

Table 2. Adjustment of Unit Capacity Values to Account for Reliability

While these capacity value adjustments are small, they apply to the entire dependable capacity of the plant, so they result in substantial benefits. Table 3 summarizes the calculation of the increase in capacity benefits attributable to both the increases in dependable capacity and increases in reliability.

Subtracting out the previously calculated benefits for the gains in dependable capacity, the gain in capacity benefits as a result of improved reliability is (\$1,150,000-\$600,000) = \$550,000 for the new runners alone and (\$6,050,000-\$4,900,000) = \$1,150,000 for the combined plan of new runners plus rewind.

Flexibility Benefits

An additional area where benefits might accrue to powerplant rehabilitation is in the area of flexibility - the ability of a powerplant to come on-line quickly and to respond rapidly to changes in load. An example might be a plant with aging Kaplan units which have deteriorated to the point where the turbine blade adjustment

Case	Depend Cap (MW)	Cap Val ($/kW-yr)	Total Ben ($1,000)	Incr Ben ($1,000)
Existing	185	117	21,650	--
New Runners	190	120	22,800	1,150
+ Rewind	226	123	27,700	6,050

Table 3. Capacity Benefits Attributable to Improved Reliability

mechanism can no longer be operated reliably. In such cases, the blades may have to be welded in a fixed position, so that they lose their ability to follow load. Rehabilitating the units would restore this capability, and this in turn would generate some benefits which could be used to help support the investment in the rehab work.

Unfortunately, while it is widely agreed that flexibility benefits are an important hydro project output, it is difficult to quantify such benefits. EPRI and others have done some work in this area (EPRI, 1984, and US Water Resources Council, 1981), but so far an accepted procedure for quantifying flexibility benefits does not exist. However, if a proposed rehab project does improve a project's flexibility, this should at least be addressed qualitatively in the rehab project feasibility report.

Total Gain in Benefits

The total power benefits attributable to the combined runner replacement/ stator rewind plan would be as follows:

Energy Benefits	=	$2,550,000
Capacity Benefits	=	$6,050,000
Total Benefits	=	$8,600,000

Last-Added Test

Accepted economic practice requires that separable components of multi-component plans be incrementally justified on a last-added basis. For instance, the example rehab plan includes two components, and for the plan to be economically feasible, both runner replacement and generator rewind would have to be individually justified on a last-added basis.

Last-added analysis refers to a comparison of (a) the incremental benefits gained by one component of a plan on a last-added basis, to (b) the incremental costs

of including that component in the plan. The last-added benefits for a component are determined by deducting (a) the benefits of a plan with that component excluded from (b) the benefits of the plan with all components included. Again referring to the example, the last-added benefits of the generator rewind would be the benefits of the total plan minus the benefits of runner replacement alone. A similar process would be followed to determine the incremental benefits of the runner replacement.

Conclusions

It has been shown that powerhouse rehab programs can increase output in a number of different ways. Techniques are now available for quantifying most of the economic benefits resulting from these increases in output for major rehab projects - those which involve turbines, generators, and other elements of the power train. Properly identifying and quantifying these benefits is a key element in establishing the feasibility of such projects.

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Use of Event Trees and Economic Models In Evaluating Hydroelectric Rehabilitation Projects

Patricia Obradovich[1]

Abstract

U.S. Army Corps of Engineers (Corps) guidance requires the use of engineering reliability analysis in conjunction with economic analysis for the evaluation of potential rehabilitation projects. This paper focuses on the use of "event trees" to facilitate the planning process and the use of an economic model which incorporates engineering reliability, operations and maintenance data, alternative project costs, and system production costs to determine rehabilitation plan selection and optimization.

Introduction

As discussed in *Major Rehabilitation Program, An Overview* (Chapman, 1993), aging Corps infrastructure will require significant rehabilitation investments over the next several years just to maintain current levels of operation. Given a static Corps' Operation and Maintenance (O&M) budget, and a growing need for rehabilitation and modernization, the Corps, the Office of Management and Budget (OMB) and other Federal decision makers are faced with the ever present economic dilemma of allocation of scarce resources. With an increasing number of competing projects vying for funding, increased emphasis is being placed on providing decision makers with better information on the need, type, and timing of major rehabilitation improvements.

[1]Patricia Obradovich, Regional Economist, Portland District, U.S. Army Corps of Engineers, PO Box 2946, Portland, Oregon 97208-2946, (503) 326-5268

Recent Corps guidance for submission of major rehabilitation reports highlights these needs by requiring reliability analysis in conjunction with detailed economic analysis. Engineering reliability analysis coupled with the traditional "engineering judgement" offers a more effective and objective way of identifying future events and consequences. Detailed economic studies including risk and uncertainty analysis provides decision makers with a more comprehensive picture of the range and likelihood of the economic consequences of any particular project proposal.

The guidance provides tools in the form of procedures for engineering condition analysis, use of event trees and prescription of other planning requirements for use in major rehabilitation studies.

The following discussion describes some of the tools that were used in the economic analysis for the Bonneville Major Rehabilitation Evaluation Report (Corps of Engineers, March 1992). In particular, use of event trees and the economic spreadsheet model used in the evaluation will be described.

Event Trees

An event tree is simply a diagram of the potential events and outcomes that could occur to a given component or group of components in one time period or in subsequent time periods.

Event tree diagrams are used to identify possible occurrences of satisfactory or unsatisfactory performance and their consequences, given specific events. For example, a mechanical/electrical component such as a turbine runner or a generator, during any time period, may be fully operational, out of service from a prior period, or exhibiting "unsatisfactory performance", i.e. failure.

These possible events or "branches" of the tree identify all of the pathways that may occur during each time period. The event tree is developed for each component to be evaluated for each time period of the analysis.

The consequences of each pathway are also identified. The consequences may consist of: changes in system hydropower generation costs due to unit outages or changes in unit generating efficiencies; increases or decreases in operation and maintenance costs; or changes in repair or replacement costs.

The event tree also facilitates coordination of the engineering reliability analysis with the economic evaluation. In the Corps' planning framework, the event tree assists in developing a clear definition of the "without-project condition". For major rehabilitation studies, the without-project condition is a description and evaluation of the consequences that are expected to occur during the period of analysis in the absence of rehabilitation. Use of event trees requires planners (and project engineers) to graphically depict what is expected to happen to various

components in any given time period. This process helps clarify critical elements and possible solutions. It highlights any apparent data gaps and serves as a road map for building the economic spreadsheet model. An example of the basic event tree for Bonneville First Powerhouse turbine runner is depicted below.

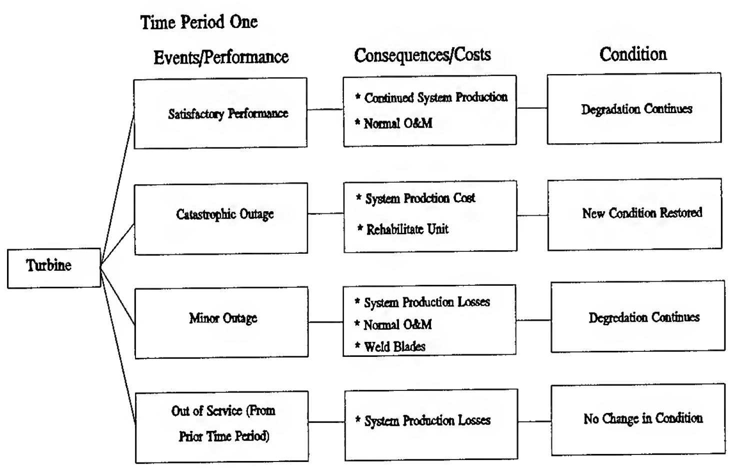

Figure 1. Bonneville First Powerhouse Turbine Event Tree

Bonneville Project Overview

In order to discuss the economic spreadsheet and the modelling process, a brief overview of the Bonneville project is warranted.

The Bonneville project consists of two powerhouses. The first powerhouse contains 10 Kaplan units which were placed into service beginning in 1938. The total rated capacity is 532 megawatts (MW). The second powerhouse was placed into service in 1981. It includes 8 Kaplan units with a total rated capacity of 558 MW. Average discharge in the Columbia River at Bonneville can support an estimated 760 average MW of generation. Flows vary considerably by month; peak flows between May and August can require full use of both powerhouses. Currently, use of the second powerhouse is restricted due to fish passage problems. It is estimated that these problems will be corrected in the next few years allowing for full use of both powerhouses when conditions permit. Sequential loading of the units in the first and second powerhouses is dictated by a number of operational

conditions including fishery considerations, station service requirements and specific voltage line requirements.

Problems at the first powerhouse consist of turbine blade cracking and breakage and generator coil degradation. Over the past 10 years the turbine blades have exhibited increased cracking. On three separate occasions, pieces of blades have broken off. An enhanced maintenance program was instituted including more frequent inspections and welding repair. This has prevented further breakage. However, blade cracking continues to increase at an accelerated rate. Deterioration of coil insulation causes generator outages. Significant outages have lead to replacement of five of the original 10 generators. Unsatisfactory performance of either the generator or turbine blade will cause a unit outage. In addition, it is estimated that the units have experienced an efficiency loss of 4.5 percent from their original condition.

The Economic Model

In its most simplistic form, the economic model developed for the Bonneville rehabilitation analysis could be described as a basic accounting spreadsheet. In its final evolution it spanned more than 2 megabytes of computer disk space, devoured hundreds of hours of computer time and dominated up to a dozen personal computers while running and compiling late night simulations.

Given the large number of hydropower generating units at the Bonneville project and the complex interrelationship between operating components, the spreadsheet model was first created to mirror the event tree diagram for the without-project condition. This incorporated both the physical and economic consequences of possible events and the engineering reliability analysis for each component. In addition, a Monte Carlo simulation procedure was used to calculate variance and expected values.

Monte Carlo simulation is a process in which random numbers are generated from a range of possible values, usually between zero and one, with any number in the range having an equal likelihood of occurrence. Each random value is input into the spreadsheet and the spreadsheet is recalculated to arrive at an associated outcome. Each random trial or iteration of the spreadsheet represents an independent "what if" game. By generating hundreds, or in some cases, thousands of "what if" games, Monte Carlo sampling will generate the input distribution and the entire range of potential outcomes.

Model Requirements

Basic functional requirements were established for the model. These requirements allowed for flexibility in the analysis, incorporation of basic assumptions, and the ability to change parameters as needed. Some of these requirements are described below.

- The model must accurately reflect the without- project condition. The without-project condition establishes a base condition from which all other alternatives are to be evaluated.

- The model must be flexible enough to evaluate a full range of alternatives. Alternatives considered in the analysis included: enhanced maintenance, use of spare parts, a full array of rehabilitation scenarios and subsequently, appropriate timing of any rehabilitation strategy.

- The model must distinguish between individual operating components, physical and economic consequences of various alternatives, and the timing of events.

- The model must be able to incorporate incremental analysis of each unit and its separable components.

- The model must account for a project life (in this case, 35 years) and for near-term events that could impact future rehabilitation strategies. Thus, the period of analysis selected began in 1992 and ended in 2035.

- The model must be able to incorporate the engineering reliability and risk and uncertainty analysis for each time period and each functional component under evaluation.

- For each alternative, the model must be able to incorporate routine and non-routine O&M costs for each component over the period of analysis.

- The model must be able to account for changes in unit efficiencies with various rehabilitation scenarios.

- The model must be able to incorporate the consequences of events and repair/rehabilitation scenarios in terms of changes in hydropower system production costs and alternative construction costs. Each alternative produces different hydropower outputs, system production costs and O&M costs.

- The model must be able to accommodate other economic calculations such as present valuation and amortization of costs and incorporation of interest during construction.

Model Operating Characteristics

For each alternative considered, the spreadsheet was modified to simulate the specific engineering, operational and economic consequences relative to the alternative. Monte Carlo simulation techniques were incorporated into the spreadsheet. This approach uses random number generation to compute an expected result given a combination of probabilities and events. The program sums the results of multiple iterations of the simulation and produces expected values and variance. Each simulation included a minimum of 300 iterations. In some cases, up to 5,000 iterations were computed.

Separate simulations were conducted for the without- project and for each alternative considered in the analysis. Simulations included: rehabilitation of one to 10 turbines; rehabilitation of turbines with the addition of generators; and rehabilitation of one to 10 generators. The appropriate timing for rehabilitation was also evaluated. It was determined that an "enhanced maintenance" strategy was already being implemented in the without-project condition. A spare parts alternative was considered, but had only limited application to generators. Incremental analysis of the alternatives allowed for optimization of the number of turbines and generators to be rehabilitated. In all, more than 500 combinations of alternatives were evaluated in the analysis.

This process even considered the physical condition of the individual units and the potential sequencing of repairs.

For each simulated outage, consequences, in the form of costs resulted from increased frequency of repair, increased maintenance effort, and having to resort to more expensive means of energy production.

Incorporation of Physical and Economic Consequences

The first few columns of the spreadsheet model account for the engineering reliability analysis. The engineering reliability analysis establishes the probability of unsatisfactory performance for each component for current and future conditions. This probability, over time, is inserted for each year in the modelling sequence. Current conditions and probabilities of unsatisfactory performance vary for each individual turbine and generator.

Within each iteration, a random number is generated for each component in a given time period. Based on the probability of unsatisfactory performance in that time period, the unit either incurs an outage or continues to operate. For example, if the probability of unsatisfactory performance for turbine unit number one in the year 1993 is 2.19 percent, then any random number generated between zero and one that is less than .0219 will cause an outage to occur, any number greater than .0219 will indicate that the unit is still available for operation. If the unit remains operational, then the probability of unsatisfactory performance in the next time

period increases. A random number is generated for each successive time period and the consequences are recorded. Should a unit incur an outage, depending on the alternative being modelled and the type of outage, the unit will either be repaired or rehabilitated. If the unit is repaired, then the probability of unsatisfactory performance in each successive time period continues to increase. If a unit is rehabilitated, then the probability of unsatisfactory performance is returned to a "new" condition as the equipment is considered to be restored.

Types of Unsatisfactory Performance

For this analysis, two types of unsatisfactory performance are evaluated for turbines and generators. Each type has a different probability of occurrence. The first type is considered to be a catastrophic outage. For a turbine, this type of outage could occur if there is significant cracking near the turbine hub. This would require full rehabilitation of the turbine unit. The second type of outage is more likely and less debilitating. This outage mode consists of blade breakage. In the past, sections of the blades weighing as much as 8 1/2 tons have broken off the units. This necessitates dewatering of the unit and rewelding the broken blade.

For each type of unsatisfactory performance, outage times and costs for repair are computed. For the turbines, blade repair is estimated to take 8 months at an estimated cost of $650,000. For a catastrophic outage, the unit is estimated to be out of service for a period of 34 months at a repair cost of $5,500,000.

For each alternative considered, routine annual O&M costs are also estimated. Under existing conditions, the turbine units are dewatered and inspected once every few months. If a unit is rehabilitated, inspections are assumed to decrease in frequency with a resulting reduction in O&M costs. In addition, the time associated with inspections and routine maintenance is also accounted for in each iteration.

Subsequent columns in the spreadsheet sum all unit outage for a given year. Sub-routines are incorporated in the model to prevent double counting of outage time if two interrelated components are out concurrently. If the unit is considered to be out of service in excess of 12 months, outage times are carried over into the next time period.

Additional columns sum O&M, repair and rehabilitation costs for any given year. Again, sub-routines are used to prevent double counting of normal maintenance costs if the unit is considered to be out of service for an extended period of time.

Columns are added to the spreadsheet to account for specific alternatives and conditions. For example, in one alternative including a planned sequence of rehabilitation, if a unit outage occurs within a year of the planned rehabilitation, the unit is not repaired or returned to service prior to the rehabilitation. This is because

at Bonneville project it proved to be more cost effective to leave the unit off line then to return it to service and then shut it down later for a permanent rehabilitation.

Another column accounts for the whether or not existing spare parts are available for a given unit. At the first powerhouse, a spare generator winding is available for unit number one. In any simulation, if unit number one's generator experiences a catastrophic outage, the existing spare part is assumed to be put into service.

Cost of Replacement Power

For any alternative in a given year, total unit outage time is summed for the powerhouse. The total unit outage is then used to determine alternative system production costs that would be required to meet the needs of the Pacific Northwest system given the loss of hydropower production capability. The costs of replacement power are generally assumed to come from thermal generation.

For the Bonneville project, development of the system production costs for each alternative entails a complicated modelling process involving the use of three interrelated models and hundreds of computer simulations. These models are referred to as hydro allocation model (HALLO), hydro seasonal system regulation model (HYSSR) and systems analysis model (PC-SAM). This modelling process occurs outside of the economic spreadsheet. Only the results of these analyses are included as inputs to the economic model.

This system production cost modelling includes the following sequence. First, the proper hydropower unit loading order for both Bonneville powerhouses is determined. This accounts for existing operational requirements and assumptions about the future availability of the second powerhouse. This is necessary as the unit loading sequence, coupled with individual unit efficiencies determine the amount of production that will occur in any given period. In the without-project condition, as units begin to experience outages with increasing frequency, production declines and other sources of power must be brought on line to meet demand. In each of the alternatives evaluated, as units are rehabilitated, or returned to service, hydropower output will increase. Thus, the use of replacement power from more costly sources will decrease resulting in a savings to the region in system production costs.

Incorporation of the loading order is conducted in HALLO. HALLO ensures that the units are loaded in the proper sequence for the without and all alternative project conditions. HALLO allows for specification of individual unit sizes and efficiencies. When unit efficiencies change due to further deterioration or rehabilitation flows allocated to each unit also change.

The HALLO model operates as a post processor to the output of HYSSR. HYSSR simulates 50 years of water records for the Columbia River basin. It includes a number of system operational constraints and characteristics and allocates

flows to each hydroelectric project based on the flows available in a given year and seasonal power demand patterns. Allocations of flow to the Bonneville project are then sub-allocated to the HALLO model to determine Bonneville production outputs for all water years for each of the alternatives considered.

HYSSR sums hydropower production for the 50 years of record for all of the projects included in the model. This production information is then included in the third model, PC-SAM. PC-SAM accounts for energy demand and power production within the Pacific Northwest and the Pacific Southwest. First, system hydropower generation is allocated to carry as much of the load as possible. Then, thermal resources are dispatched from least cost to higher cost in order to meet system demand. Then, the cost of operating the system is computed. The amount of production from the Columbia River hydropower system and each individual project can thus impact the total system energy costs for any given year.

Using HALLO, HYSSR and PC-SAM, the without-project condition for the Bonneville project is modelled. This produces an annual system production cost assuming all first powerhouse units are available for production. Next, the without-project condition is modelled assuming that one less unit at the first powerhouse is available. Subsequent scenarios are run removing a unit at a time until all 10 units are considered to be off line. This process results in construction of a system production cost curve assuming a full range of unit availability in the without-project condition. This production cost curve is then used in the economic model to quantify the production cost consequences of unit availability for any potential combination of randomly generated unit outages.

Additional production cost curves are constructed to assist in modeling all of the potential rehabilitation and repair scenarios. As units are rehabilitated, unit efficiencies increase, hydropower production increases and system production costs decrease.

Once all of the separate cost curves and previously described input values are established, the without-project and all of the with project conditions are simulated. For each iteration of a simulation, potential outages are generated, O&M, repair and rehabilitation costs are calculated and system production costs are estimated. The economic consequences for each alternative over the period of analysis are summed and described in present values terms. Net benefits are computed for each alternative and the plan that maximizes net benefits is recommended for implementation. For the Bonneville study, the alternative that provided maximum net benefits includes rehabilitation of all 10 turbines and five of the generators. Additional statistics are generated to describe the range and distribution of values for each component.

Summary and Conclusions

Although it is difficult to convey the economic modelling process in this brief discussion, it is hopefully easier to see how the engineering reliability analysis and the project economics can be better integrated in a risk-based analysis. Use of event trees provides a graphic display of the problem components and modes of unsatisfactory performance. This, in effect highlights data needs and critical elements of the analysis. Event trees also allow for a systematic evaluation of alternatives. By comparing any proposed alternative to the without-project condition, the economic consequences can be consistently calculated and evaluated.

Use of the spreadsheet model facilitates incorporation of risk and uncertainty analysis with the engineering reliability analysis. The outcome is a presentation of benefits and costs for each alternative that accurately combines a full array of possible events.

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THE CORPS OF ENGINEERS IN HYDROPOWER DEVELOPMENT

Bradford S. Price, P.E., Member (1)

Abstract

This paper presents the role of the U.S. Army Corps of Engineers (Corps) in the development of hydropower. Topics presented will include: (1) History; (2) Corps Procedures For Evaluating And Implementing Hydropower Projects; (3) Hydropower At Corps Projects Today; (4) Development Of Hydropower At Corps Projects By Non-Federal Interests; (5) Headwater Benefits Attributable To Corps Projects; and (6) Upgrading Of Corps Hydropower Projects.

History

Corps involvement in hydroelectric power production stems from its water resources mission. In 1824, Congress assigned the Corps its first water resources task -- that of clearing snags and sandbars from the Ohio and Mississippi rivers. This initial assignment expanded to a general responsibility for navigation improvements.

Then in 1909 the Federal government acquired a dam on the St. Mary's River in Michigan. Though the acquisition was primarily for navigation purposes, the site also contained a hydroelectric power plant, the Corps' first. Congress, recognizing the potential significance of hydroelectricity to the growing nation, directed the Corps to include assessments of water power potential in its periodic surveys of U.S. waterways. At that time, development of hydroelectric facilities was conducted almost entirely by

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(1) Civil Engineer, Project Management Policy and Development Branch, CECW-LP, HQ, U.S. Army Corps of Engineers, 20 Massachusetts Ave. N.W., Washington, D.C. 20314-1000

private groups. But the Corps began its own hydropower construction in 1916 when it added a turbine and generator unit to the St. Mary's plant.

In 1918 the Corps began building the Wilson Lock and Dam on the Tennessee river in Alabama. Completed seven years later, the project contained hydroelectric generators with a total capacity of 184,000 kw. About that time, the Corps, which in 1917 had been charged by Congress with flood control responsibilities, began comprehensive water resource planning in a series of studies known as the 308 Reports. And over 50 years ago the Corps started a comprehensive hydroelectric program with the design and construction of a 518,000 kw plant at the Bonneville Lock and Dam project on the Columbia River in Oregon.

Particularly since the end of World War II, the Corps' role in hydroelectric development has grown and expanded. Many public and private groups and two Federal agencies, the Interior Department's Bureau of Reclamation and the Tennessee Valley Authority, have also developed water power resources. However, the Corps of Engineers is the nation's largest builder, operator, and sponsor of hydroelectric facilities. And, the Corps follows a fairly rigorous procedure for evaluating and implementing hydropower projects.

Corps Procedures For Evaluating And Implementing Hydropower Projects

The U.S. Congress authorizes and partially funds Corps hydropower projects. Before authorizing a project, the Congress must be convinced that it will meet a legitimate need, will be well designed, economically feasible and environmentally sound.

The process usually begins when local citizens or agencies urge their Congressional representatives to introduce legislation authorizing the Corps to study a proposed multiple purpose project. When legislation authorizing a study is passed and money appropriated, the Corps District in whose area the proposed project is located begins engineering, economic and environmental investigations.

The overall study proceeds in two phases. The Reconnaissance Phase is at Federal expense. It is a preliminary study, and among its goals are: (1) to determine if there is likely to be at least one economically feasible and engineeringly implementable alternative plan; and (2) to identify a non-Federal sponsor that agrees to contribute to costs for the Feasibility Phase of study. Planning will not proceed to the Feasibility Phase, which is more detailed and demonstrates the economic, environmental and engineering feasibility of potential projects, without a sponsor willing to share the Feasibility study costs.

A non-Federal sponsor's participation is critical for there to be a hydropower analysis. Hydroelectric power is considered for inclusion at Federal expense only when there are compelling reasons why non-Federal ownership, operation and maintenance of hydropower at a multiple purpose project are impracticable.

The District Commander, who heads the District staff, holds public meetings. Citizens are asked to comment first during the early planning stages, later when alternative plans are being considered, and finally when a specific plan is formulated. District planning also involves state and municipal officials and various Federal agencies. These Federal agencies include the Department of Energy (DOE) which would market the proposed project's power and the Federal Energy Regulatory Commission (FERC).

During the study phase, the Corps and the non-Federal sponsor evaluate the proposed project according to stringent criteria. The project study must include investigations of the adverse environmental effects on the streams on which it may be located and on the surrounding countryside. The electricity produced must fit into the overall regional power needs. The FERC'S and the DOE marketing agency's analysis of this criterion is especially important.

In, addition, the total economic benefit from the project must exceed its total cost, with benefits and costs calculated on an annual basis. A project's economic life is usually 100 years, and costs include the initial capital investment, interest over the project life, as well as estimated operation, maintenance, repair, replacement and rehabilitation (O.M.R.R.&R.) expenses. Benefits include anticipated economic value of functions and services provided by the project--navigation, flood control, recreation, water supply, downstream low flow maintenance, and hydropower. The economic value of a project's hydroelectric power production is estimated on the basis of the comparable cost of power production by the most likely alternative source of power, usually a thermal generating plant.

The cost-benefit analysis of not only the entire multiple purpose project but also of the hydropower portion alone must be favorable. The cost-benefit analysis also insures that electric power is supplied at the least possible cost to consumers and that the Federal government invests in the most economically efficient projects.

The project must also pass a financial feasibility test. This test measures whether projected revenues from the sale of power will be sufficient to recoup the costs of producing and marketing that power. While the Corps builds and operates most of its dams, it doesn't sell the power. Under Federal law, power generated at Corps projects is marketed by the Department of Energy to public bodies, power cooperatives and private utilities. Although electricity is not sold directly to the consumer, the underlying goal of all Corps hydroelectric projects is to provide power to

consumers at the lowest possible rates. Rates are set by the marketing agency and approved by the FERC.

Five Department of Energy agencies sell power from Corps projects. These marketing agencies, each serving a different part of the country, are the Alaska Power Administration, Bonneville Power Administration, Southwestern Power Administration, Southeastern Power Administration, and the Western Area Power Administration. Future revenues are estimated by the marketing agency responsible for the distribution and sale of the power. If constructed at Federal expense, project costs allocated to power must be recovered within 50 years.

With the help of citizens and other government agencies, the Corps and the non-Federal sponsor evaluate the proposed multiple purpose project according to these criteria. Results of the study are incorporated into the final project plan. The District Commander prepares a detailed report on the planning phase of the proposed project. He also prepares an Environmental Impact Statement (EIS) outlining the facility's anticipated effects on the physical and social environment. After the report and the EIS are reviewed within the Corps, they are submitted by the Chief of Engineers to the Secretary of the Army for approval. Before recommending projects, the Secretary seeks concurrence of the Office of Management and Budget. Finally, the report and EIS are submitted to Congress, where the House and Senate Public Works committees hold hearings on the proposal.

If Congress finds the proposed project to be to the nation's benefit, it authorizes construction and appropriates money. Non-Federal sponsors must arrange financing for their part of the project costs so that funds will be available as design and construction progress. The financial and other requirements of the non-Federal sponsor and the Secretary of the Army acting for the Federal government are outlined in agreements that are executed by both parties. Then Corps District officials again take responsibility, first making final engineering designs and later undertaking actual construction.

The installation of hydropower at Town Bluff Dam, near Jasper, Texas, is an example of how most hydropower will be financed at Corps projects in the future. Town Bluff Dam is a Corps flood control project that was completed in 1958. The energy situation in that area in the 1970's early 1980 time period indicated that adding a hydropower capability to the dam was economically feasible. After the local sponsors chose the Corps to construct the hydropower project, representatives from the Sam Rayburn Municipal Power Agency and the Southwestern Power Administration signed three agreements with the Federal government regarding financing construction and arranging for power sales. One of the provisions of the agreements was that the local sponsors would pay the full amount of estimated construction costs prior to the beginning of construction. Local sponsors would also be responsible for annual O.M.R.R.&R. costs.

Hydropower At Corps Projects Today

Today, the Corps operates 75 projects with a total capacity of 20.72 million kw. The Corps has about 30 percent of the nation's hydroelectric capacity and 3.0 percent of the nation's total electric power. About two-thirds of this capacity is in the Pacific Northwest, where the Corps provides nearly one-third of that region's electricity. Water power is so abundant in that part of the country that hydroelectric facilities developed by other agencies and private utilities comprise the major portion of the remaining electric power production.

In 1991, Corps' facilities produced 83.9 billion kwh (1 kwh=production of 1 kw for 1 hour) of electric energy. This energy production was equivalent to the output of about 19 average size nuclear plants. To produce that much energy from the nation's nonrenewable sources would have required burning approximately 30 million metric tons of coal, 29 billion cubic meters of natural gas or 20 billion liters of oil. Revenue from energy sales in 1991 was approximately $500 million.

Development Of Hydropower At Corps Projects By Non-Federal Interests

There are also 63 Corps projects at which hydropower has been developed by non-Federal interests through licensing activities administered by the FERC under the Federal Power Act. These projects have an installed capacity of approximately 2.7 million kw. Additionally, there are 6 projects that are approved for construction and another 24 that are at various stages in the design and approval process.

The non-Federal developer must evaluate the effect of proposed hydroelectric power plant construction on Corps water control management responsibilities such as flood control, navigation, water supply, low flow augmentation, water quality, and other purposes. The Corps will make the appropriate recommendations to the FERC for safeguarding all of the above functions that may be affected. The non-Federal developer will also be subject to other general requirements including:

1) Verification of compatibility with authorized purposes at Corps projects may, under certain circumstances, require physical and/or mathematical modeling, the cost of which will be borne by the non-federal developer.

2) Full power potential of the site must be considered in planning, design and construction of the power plant.

3) Design, construction and operation of all power facilities which would affect the structural integrity and operational adequacy of the Federal dam, including construction procedures and sequence, must be approved by the Corps.

4) In the interest of multiple-purpose water management, the Corps requires a signed memorandum of understanding between the prospective

developer and the Corps specifying the operational procedures and power rule curves consistent with overall project management objectives and efficient system flow regulation.

5) The developer must reimburse the Federal government for the use of lands, facilities and an appropriate part of the costs of existing Federal projects which makes the installation of power feasible. Assessment of these costs and development of charges will be made by the FERC.

6) Reimbursement to the Federal government will be required for any additional construction costs incurred by the government as a result of installation of the power facilities.

7) The developer will provide power, free of cost, to the United States for operation and maintenance of navigation facilities at the project site.

8) The developer will furnish, operate and maintain adequate lights, signals and protective warning devices to provide for safe navigation and the safety of persons using the Federal project.

9) In compliance with Section 404 of the Clean Water Act (33 U.S.C. 1344), a Department of the Army permit is required for any discharge of dredged or fill material, including activities associated with hydropower development, into the waters of the United States.

Headwater Benefits Attributable To Corps Projects

In addition to developing hydropower at Corps projects, non-Federal developers have developed projects downstream from Corps projects. The operation of some of the Corps projects contributes to increased generation of hydropower at these downstream non-Federal projects.

For Corps multiple purpose reservoir projects, water is stored during high flow periods and released during low flow periods for navigation, hydropower, water supply, or other purposes at rates higher than would have occurred naturally. For Corps single purpose flood control reservoirs, water is stored during high flow periods but is usually released soon after the threat of flooding has subsided but again, like multiple purpose projects, at rates that are higher than would have occurred naturally. It is this higher flow during periods when flows are naturally lower that benefits downstream hydropower projects through increased energy generation.

The increased hydropower generation is referred to as "Headwater Benefits". The FERC under the Federal Power Act is charged with the responsibility of determining these benefits to hydropower projects downstream from Corps projects The FERC procedures consider factors such as the flow regime prior to and after construction of the Federal project when determining the increased output of power and/or energy from the downstream project.

The costs that were incurred by the Federal government in constructing and operating the Federal project upstream are also considered in the

evaluation. The value of the power gains at each downstream non-Federal project together with the value of benefits for other purposes, such as flood control, navigation, irrigation, and recreation, resulting from the upstream Corps project is used in allocating the annual costs of the Corps project to power and other purposes. The average annual costs of interest, maintenance, and depreciation thus apportioned to power is then considered by the FERC in assessment of headwater benefits to downstream beneficiaries. It should be noted that the annual costs thus derived are often considerably less than the benefits from the increased power and energy generation to the downstream beneficiaries.

Headwater Benefits for 1991 due to the operation of 98 Corps projects are estimated to be approximately $7.1 million which will be deposited into the Federal Treasury. Cumulative benefits through 1991 amount to $70.6 million. The increased average annual energy generation to hydropower projects downstream from the Corps projects totals approximately 5.3 billion kwh. The distribution of energy gains and the number of projects by Corps of Engineers District/Division and river basin is listed in Table 1.

Upgrading Of Corps Hydropower Projects

One other way that the Corps contributes to hydropower development is through the upgrading of its projects. The Corps convened a task force in June 1992 to develop a strategy to guide future investments needed to operate, maintain, rehabilitate and upgrade existing hydropower facilities. A new strategy for managing hydropower investments is needed because the long term outlook suggests an accelerated need for capital investments to rehabilitate and, perhaps, increase the capacity of existing Corps hydropower facilities

The Corps has invested nearly $8 billion in existing hydropower facilities since the first project was brought on line with another $85 million spent on maintenance and rehabilitation in FY 91. Over one-half of these facilities are at least 30 years old which suggests a need for capital replacement over the next ten years.

The Federal deficit will tend to restrict any increases in Corps funding to meet these needs. Further, current Corps policy emphasizes cost-sharing requirements stemming from the passage of the Water Resources Development Act of 1986 which requires 100% up-front financing for all new hydropower projects. The Federal deficit and cost sharing policy may have implications for financing of capital investments for existing hydropower facilities that will have to be addressed in the development of the strategy for managing hydropower investments.

TABLE 1-HEADWATER ENERGY GAINS DUE TO CORPS PROJECTS

CORPS DISTRICT/ DIVISION	RIVER BASIN	NUM. OF CORPS PROJECTS	AVG. ANN. ENERGY GAINS MWH*
NEW ENGLAND	MERRIMACK	4	1,755
NEW ENGLAND	FARMINGTON	1	1,864
NEW YORK	WINOOSKI	2	2,202
BUFFALO	GENESEE	1	7,530
PITTSBURG	OHIO/KANAWA	18	SUM OF ALL
HUNTINGTON	OHIO/KANAWA	30	PROJECTS
LOUISVILLE	OHIO/KANAWA	6	IN BASIN
			84,721
WILMINGTON	ROANOKE	2	390,985
SAVANNAH	SAVANNAH	3	13,010
MOBILE	ALABAMA	2	94,055
MOBILE	CHATTAHOOCHE	2	82,552
ST. PAUL	MISSISSIPPI	6	41,538
VICKSBURG	OUCHITA	1	13,436
LITTLE ROCK	WHITE (ARK)	1	10,570
KANSAS CITY	OSAGE	4	8,336
SEATTLE	COLUMBIA	2	4,415,040
SEATTLE	WHITE (WASH)	1	4,251
WALLA WALLA	SNAKE	1	4,009
PORTLAND	WILLAMETTE	10	111,250
SACRAMENTO	KERN	1	25,890
	TOTAL	**98**	**5,312,994**

*-Energy Gains Accrue To Downstream Non-Federal Projects

Conclusion

There are 75 Corps hydropower projects with a total capacity of 20.72 million kw that generated 83.9 billion kwh of electric energy in 1991. The revenues from the power and energy from these projects amounted to approximately $500 million in 1991. Planned additions to these projects will bring the ultimate capacity to 24.58 billion kw. Additionally, the operation of 98 Corps multiple and single purpose projects results in average annual energy gains to downstream non-federal hydropower projects totaling approximately 5.3 billion kwh. Resulting headwater benefit revenues for 1991 were estimated to be $7.1 million.

The Corps of Engineers has been a leader in the development of hydropower throughout the years. The numbers attest to that fact with the Corps generating about 30 percent of the nation's hydroelectric capacity and 3.0 percent of the nation's total electric power. And, the Corps will continue to lead with its: (1) continued development of new hydropower projects in cooperation with non-federal sponsors; (2) development of a long term investment strategy to rehabilitate and, perhaps, increase the capacity of existing Corps hydropower facilities; (3) continued support of the development of hydropower by non-Federal interests at Corps single and multiple purpose facilities; and, (4) continued support of the FERC in their Headwater Benefits investigations.

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LAKE ELSINORE PUMPED STORAGE PROJECT

S. T. Su, Fellow, ASCE

Abstract

The proposed Lake Elsinore Pumped Storage Project is located near the town of Lake Elsinore in Riverside County, California. Planning and power studies were conducted to determine the optimal installed capacity, energy storage, and related physical dimensions of the project facilities. The proposed pumped storage development will be centered on a project with an installed capacity of 240 MW.

This paper presents the results of a planning investigation to establish the project feasibility based on future targeted system needs and environmental enhancement and mitigation measures. Special provisions are provided to integrate water quality enhancement at Lake Elsinore with project design and operations.

Introduction

The Cities of Anaheim, Azusa, Banning, Colton and Riverside need peaking capacities to meet their electric power demand, projected in the late 1990's and beyond. Pumped storage is a proven technology for converting off-peak energy to peaking capacities. The economic criteria of developing a pumped storage project generally require a site with relatively high head and available reservoir locations. Lake Elsinore site appears to possess the suitable topographic features to support a pumped storage project.

Vice President, Water Resources, Harza Kaldveer Consulting Engineers, 425 Roland Way, Oakland, Ca. 94621

Power System Needs

Power studies were conducted to evaluate the potential electric power resources that would take advantage of the Cities' surplus baseload capacity by storing inexpensive, off-peak energy in order to reduce on-peak capacity and energy requirements. These resources, called pumped storage units, pump and store water during the off-peak period and then release the water during the on-peak period to generate electricity.

Pumped storage units are used by various utilities to mitigate the effects of daily peaking problems. The southwest region of California, however, has few sites that can be utilized for pumped storage projects, either because of insufficient or varying water supplies or an unacceptable elevation between the upper and lower reservoirs.

Site Potential

The project location and its general arrangement is shown in Figure 1. The energy potential of the site is a direct function of the product of the head differential between the reservoirs and the amount of water cycled between the upper and lower reservoirs.

For the Lake Elsinore project, the site potential is controlled by the head differential of approximately 1,500 feet and the smaller of (a) the lower reservoir size and allowable fluctuation and (b) the capacity of any upper reservoir that can be sited on the Elsinore Mountains.

Lower Reservoir - The present lake surface has an approximate area of 4,000 acre-feet. The lake surface is expected to be reduced to about 3,000 acre-feet when the planned development and environmental improvements are implemented. If the lake is allowed to fluctuate about 8 inches for pumped storage operation, the cycling capacity would amount to 2,000 acre-feet.

Upper Reservoir - A number of upper reservoir sites are available on the Santa Ana Mountains. The most favorable site appears to be the one that can be created by the construction of a dam across Morrell Canyon, for the following reasons:

- It has enough capacity to match the required storage up to about 2,000 acre-feet.
- It can be built by the construction of a 120-foot high dam.
- The upper reservoir would be in close proximity to Lake Elsinore, the lower reservoir, resulting in a relatively short water conductor to connect the reservoirs through the power plant. Most other upper reservoirs would not have sufficient reservoir capacity that could be created with the construction of a similar size or height of dam, nor would be in the same short distance from the lower reservoir.

Site Hydrology

Elsinore Basin is located in Riverside County in the southern part of California. It is approximately 60 miles southeast of Los Angeles and some 25 miles inland from the Pacific Ocean. The Basin has an area of about 42 square miles, of which 62 percent is valley floor land and 38 percent is hills and mountains. About 30 percent of the valley floor is subject to inundation by Lake Elsinore. The basin trends northwesterly and is bordered on the southwest by the steeply rising Santa Ana Mountains. These mountains reach an elevation of 5,696 feet on Santiago Peak, while the overflow elevation of the lake is 1,260 feet. The northeast border of the basin is defined by a range of comparatively low hills. Both to the northwest and southeast the boundaries are less pronounced, the land rising gently until low drainage divides are reached.

San Jacinto River, the largest stream in Elsinore Basin, originates high in the San Jacinto Mountains and flows northwesterly through San Jacinto Valley, and southwesterly through Perris Valley, before entering Elsinore Basin in Railroad Canyon. All other streams draining into Elsinore Basin are short in length, and their aggregate flow represents only a minor part of the total surface inflow. Most important of these are the streams draining Leach Canyon and McVicker Canyon in the Santa Ana Mountains, at the northwest end of the basin. Much of the discharge from minor tributaries percolates to ground water before reaching Lake Elsinore.

Elsinore Basin is a closed drainage system, except on those rare occasions when Lake Elsinore overflows into

Temescal Wash, a tributary of the Santa Ana River. There has been significant water loss by evaporation from the surface of Lake Elsinore. Monthly evaporation rates at Lake Elsinore varies from 7.9 inches in August to 1.6 inches in February. The average annual evaporation rate is about 56 inches.

The regimen of seasonal runoff in the San Jacinto River entering the lake is extremely erratic. Large natural variations in tributary mountain and valley runoff to the San Jacinto River are further accentuated because low to moderate stream flows percolate to ground water in San Jacinto Basin. The intermittent character of inflow to Lake Elsinore, together with the shallow lake's high evaporation rate, cause the surface elevation to fluctuate.

Site Geology

The project area is characterized by Lake Elsinore, in its linear northwest-trending valley, and Elsinore mountain which borders it to the southwest. Bedrock in this mountainous area consists of plutonic igneous rocks that are mid-Cretaceous in age and range in composition from mostly granodiorite to gabbro. The Elsinore valley is filled with a thick succession of poorly consolidated sediments, mostly sand and silt.

The Elsinore valley is a down-faulted trough, or graben, separated from the bordering highlands by major northwest-trending faults, principally the Willard and Wildomar faults. These belong to the Elsinore fault zone which extends some 125 miles from the Mexican border to the northern end of the Santa Ana Mountains and is part of the active San Andreas fault system.

Pumped Storage Development

As shown in Figures 1 and 2, the project consists of the following principal features:

Dam and Appurtenant Works - The dam would be constructed in a narrow section of the Morrell Canyon Creek. The dam would be about 120 feet high above the streambed. It would be of the concrete-face rockfill type, since rockfill can be readily obtained from the granodiorite at the site. The dam would be founded on competent rock which is expected to be within a short depth from the

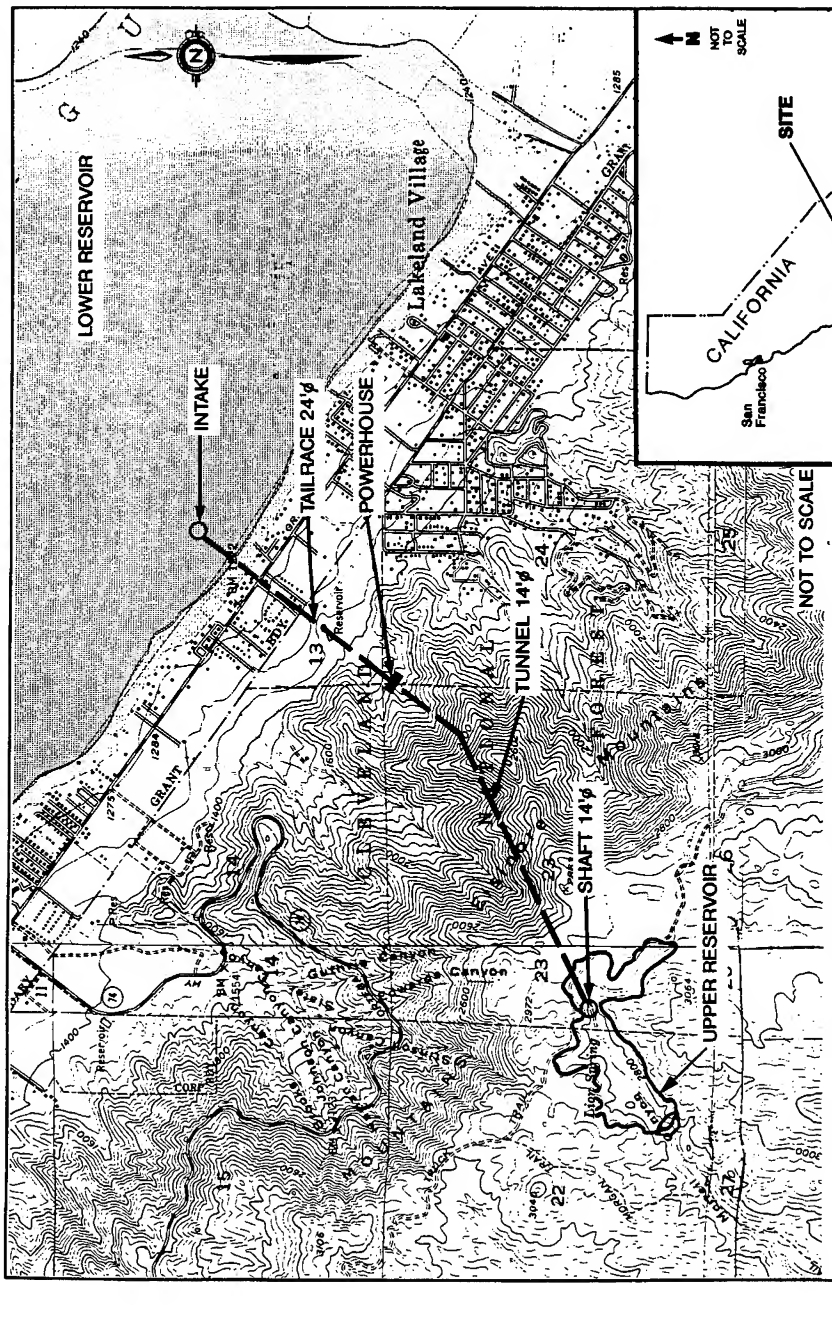

FIGURE 1 – PROJECT LOCATION AND GENERAL ARRANGEMENT

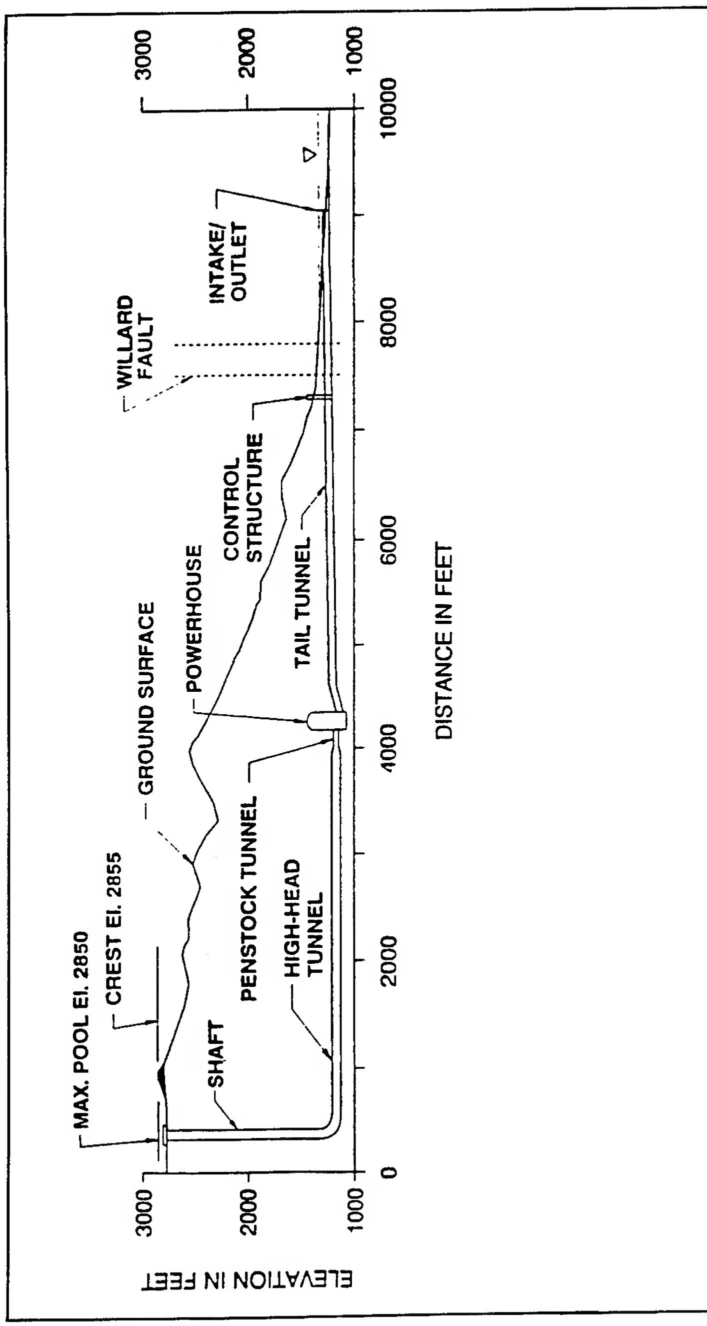

FIGURE 2 - PROJECT PROFILE

surface. A combined spillway and low-level outlet works structure would be constructed. The spillway would be sized to provide sufficient capacity to pass the maximum pumping discharge, in the event of inadvertent over-pumping of the power plant. The capacity would be sufficient to handle the maximum overflow requirement under the maximum spillway design flood inflow. An appropriate amount of surcharge would be provided in the dam to route the spillway design flood. The outlet works would be provided with low-level outlet valves to pass required flows under normal operating conditions to maintain the flow down the canyon.

Approach Channel - The approach channel would be an excavated canal designed to pass flow in both the generating and pumping directions. The invert of the channel would be set at elevation 2750 to permit passing the required power plant discharge at the minimum reservoir level.

Upper Intake/Outlet - The upper intake/outlet would be of the morning glory type, provided with a set of fixed trashracks and bulkhead gates which would permit the intake to be closed to service the high-head water conductors without emptying the upper reservoir.

High-Head Water Conductor System - The high-head water conductor system would be vertical shafts and horizontal tunnel excavated in rock and lined with concrete. The horizontal tunnel would terminate in a concrete manifold that would be divided into three penstock tunnels which would be concrete and steel lined. The entire high-head water conductor system is expected to be in relatively sound granodiorite, which should be massive and competent. The rock mass surrounding the water conductor is expected to resist the internal water pressure. However, in the vicinity of the powerhouse cavern, the penstock steel would be designed to resist the bulk of the internal water pressure.

Power Station - The power station would be underground, located far enough in the mountain to be in the massive granodiorite. The power station would be about 70 feet wide, 160 feet high, and 350 feet long to consist of three unit bays containing the pumping/generating units and an erection and service bay. Access would be by means of a near horizontal access tunnel to one end of the power station and a vertical shaft from the other

end. Each of the three pump/turbines would be of the vertical, reversible Francis type, rated to produce 80 MW at the minimum operating head. The units would operate in the pumping mode when they rotate in the opposite direction. They would be designed to pump approximately 90 percent of the generating discharge over the complete cycle of operation. The units would be set at about 120 feet below the minimum water surface in Lake Elsinore to provide sufficient submergence of the machinery. The generator/motor would be direct connected to the pump/turbine and rated to match the maximum output and input of the pump/turbine. Each generator would be connected to a power transformer located in a vault between the units, and provided with fire protection equipment. High voltage busses would be placed in the access tunnel to the switchyard on the surface. The power station would be provided with an appropriate type of starting equipment to start the units in the pumping mode.

Low-Head Water Conductor System - The low-head water conductor system begins with an extension of the draft tube from each of the three pump/turbines to merge into a single tailrace tunnel. The tailrace tunnel will terminate at a combination tunnel portal and control structure. At this point the tunnel will be connected to a cut-and-cover conduit that extends to the Lake Elsinore to the lower intake outlet. The tailrace tunnels would be concrete lined. The control structure would be built in an excavated vertical shaft, and would be provided with bulkhead gates to permit unwatering of the tunnel. The cut-and-cover conduit would be built of reinforced concrete. The lower intake would likely be a morning glory type intake structure with an concrete cover to suppress vortex formation. The control structure would be located in favorable rock, but the rest of the tunnel will likely encounter weathered rock or rock subjected to service faulting. The cut-and-cover section would likely be founded on unconsolidated soil and may require special treatment during construction.

Project Performance - The project has been designed with conventional concept, and the project features have been dimensioned according to current engineering practice. Furthermore, the water conductors are relatively short in comparison with other pumped-storage projects. Water conductor length affects head loss and therefore operating efficiency. On this basis, the project is

expected to have a fairly high cycle efficiency on the order of 75% (ratio of energy output and input, measured at the high voltage side of the power transformer).

Environmental Considerations

Among the expected environmental impacts, there are two significant environmental benefits that this project would bring in to the community. The first is the water quality enhancement at Lake Elsinore due to the improved circulation and aeration created by the pumped-storage operations. Lake Elsinore, as is with the current conditions, is oxygen-deficient for the fish during the summer months. Several fish kills occurred as a result of the lack of aeration.

The second benefit is the development of recreational opportunities around the upper reservoir in the Santa Ana Mountains. The reservoir could also be used as emergency water storage for fire control. The famous Santa Ana wind has been a major cause for the recurring fire threat to the local community.

The other indirect environmental benefit would be the use of non-air pollution source for power generation to meet the daily peak demand. In southern California, the air quality is a serious concern, and that any power generation facility that could avoid emission of air pollutants would be preferable.

Conclusions

The proposed Lake Elsinore Pumped Storage Project, as described above, is a feasible development to meet the future electric peak demand in southern California. Significant environmental benefits to the community would be realized by the Project through improved circulation and aeration at Lake Elsinore and by developing recreational opportunities at the upper reservoir area.

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ENVIRONMENTAL IMPACTS OF HYDROPOWER PROJECTS

Rameshwar D. Verma,[1] M.ASCE

Abstract

Hydropower is considered as a vital source of energy for meeting the world's needs particularly because it is renewable and also because it is considered to have lesser adverse environmental impacts as compared to other sources of energy. A number of hydropower projects are being planned and/or under construction in India and other developing countries of Asia. There are serious controversies about the environmental aspects of many of these projects resulting delay/postponement/cancellation of the projects. Some of these projects are discussed.

Consequences of neglecting environmental factors from the early stage of project identification to the later detailed work of project design and construction can be devastating both for the natural environment and for the human population it sustains. Proper environmental impacts of the hydropower projects if made during the planning stage and corrective measures taken for the adverse impacts will help in preventing this devastation. The most essential feature for the success of an environmental impact assessment is the degree to which environmental dimensions are integrated into all phases of project planning and design. Issues pertaining to environmental impact assessments of hydropower projects are presented and analyzed.

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[1]Department of Civil Engineering, Malaviya Regional Engineering College, Jaipur-302017. India.

Introduction

Availability of usable energy is a dominant factor in the economic development of a region. With the increasing oil prices, there is a serious crisis of energy supply in the developing world. Projections for the future are even more gloomy. Rogers (1991) observes that the global energy growth rate is between 2% and 3% annually, about 1.5 times the current rate of global population growth. If the trends continue, the developing world's commercial energy use in the year 2000 could be four times of 1980 particularly because of the needs of industrial and transportation sectors.

The modern commercial energy requirements are met primarily by coal, oil and hydropower. Associated gas and nuclear power have also a small share in the primary energy consumption. The environmental impacts resulting from the execution of energy projects and their operation manifest themselves in several forms. Thermal power projects lead to both atmospheric and water pollution. The harnessing of nuclear energy for power is riddled with safety problems of alarming proportions. There is growing concern regarding environmental impact of energy supply industries and steps are generally taken to minimize and avoid the adverse impacts by paying attention to them at the time of project conceptualization and planning.

Hydropower is an attractive energy source. It utilizes a renewable source and is based on reliable technology. It consumes no fuel and creates no air pollution. Hydropower plants are virtually inflation proof over their service lives because operating costs for existing facilities are minimal. Also hydropower plants can be brought on line much faster than other conventional power plants. Hydropower is dependent on the hydrologic cycle to provide water as runoff. The paper presents the potential of hydropower development. Environmental impacts of water power projects are reviewed and analyzed.

Hydropower Development

According to Veltrop (1991), hydropower contributed 21 per cent to total electric energy generation in the world in 1989 which is only 14.5 per cent of total hydro potential worldwide. Vast undeveloped hydroelectric resources are available in Asia, Africa, Central America, China and Russia. Based on a survey, Water Power and Dam Construction (August 1992) tabulated the world's hydro resources which give detailed breakdown of gross theoretical, technically feasible and economically feasible hydro potential. It is believed that full potential of

hydropower development in Asia-Pacific region is yet to be tapped. According to the United Nations Environmental Programme (1984) report only 4.1% of the estimated potential of 1 million MW in the developing countries of Asia-Pacific have been exploited. Afghanistan, Iran, Nepal, Pakistan, Sri Lanka and Thailand possess economically exploitable hydroelectric potential which can be developed to satisfy their total peak loads. The report further indicates that beyond satisfying national electricity needs there is the lucrative opportunity for electricity trade for those countries which have abundant exploitable hydroelectric potential with a low demand of their own. Goldsmith (1992) opines that the vast hydro resources still to be exploited show that there is a great deal of scope left in the hydropower sector, but there are many conflicting influences which can encourage or impede furtner progress. Dealing with these influences to give maximum benefit to the population will be the principal task.

Water-power development in the Indian sub-continent has been modest. India took up a number of large river valley projects soon after independence in 1947 and several multipurpose or purely hydel projects were completed and are presently being planned/constructed. The existing hydropower generation of 19000 MW is about 30 per cent of the total power availability of 67000 MW. About 85 per cent of the hydropower potential of the country is still untapped. The National Power Plan envisages an additional hydel generation of some 39000 MW by year 2000 making the hydro-thermal mix 34:66 as against the desirable 40:60 ratio. Verghese (1990) indicates that the Ganga-Brahamputra-Barak Basin in the Indian sub-continent is endowed with a vast hydroelectric potential of 200,000 to 250,000 MW. Nepal and Bhutan have huge hydroelectric reserves which constitute their largest single resource endowment and source of wealth. Singh (1992) suggests large hydropower potential available at a cheap rate in the Central Himalays of India.

Environmental Impacts

Environment is the sum total of all those physical, chemical, biological and socio-economic factors that impinge on an individual, a population or a community. These factors include rational and sustainable resource management for the welfare of present and future generations. The relationship between people and their environment is a two-way process. The environment affects people as much as people affect the environment. The protection of the environment must be an essential part

of development. The World Bank (1992) examines the interaction between development and the environment and suggests policies for sustained development.

The major environmental problems in hydropower development projects stems due to water impoundments with construction of dams. The environmental effects are experienced both at the upstream and downstream reaches and also within the dam area. The negative environmental impacts generally listed are: disappearance of organisms non-resistent to impoundments, loss of bio-diversity and wild life species, acceleration of eutrophication process, flooding of good quality lands or of historic, archaelogical or scenic sites, obstacles to upstream migration of fish, harmful temperature changes, increased salinity and sedimentation problems and resettlement of people. The United Nations, ESCAP (1983) have provided guidelines of environmental impact assessment for application to tropical river basin development. All the negative impacts of large impoundments are discussed at length in the guidelines. The author has reviewed these impacts in the context of irrigation projects separately (Verma - 1986). There are positive impacts such as improved economic development and employment opportunities of people alongwith recreational facilities due to impoundments which in many cases outweight the negative impacts. Some specific cases of the impacts appearing in the literature for hydropower projects follow. Obeng (1978) discusses the environmental impacts of four major multipurpose impoundments in Africa namely Lake Kariba on the river Zambezi in Zambia, Volta Lake on the volta River in Ghana, Lake Kainji on the river Niger in Nigeria and Lake Nasser on the river Nile in Egypt. The most serious adverse impact identified was the enhancement of conditions for the establishment of water related diseases especially bilharzia and schistosomiasis. The United Nations, ESCAP (1983) reviewed three case studies assessing the impacts of hydroelectric development for the Guatamala Energy Master Plan, Niger River project at Lokoja in Nigeria and the Nam Pong project in Thailand. Although the three assessments were found to be qualitative in many respects, they present useful information about trade-offs in negative and beneficial effects among alternatives, emphasize detailed impact identification, mitigation and monitoring and make project managers aware of the environmental interdependencies of their actions. Seaman (1972) examines the environmental and ecological aspects at small dams. He suggests that the best time to consider the total environmental impact and necessary adjustments is in the planning and design stages of dams and reservoirs. Some compromises are necessary to accommodate many interests and uses of water structures. Fish

and wild life management can be achieved by providing fish ladders and by building new hatcheries downstream of the dam and by suitable modifications in the design and construction of structures. Jai Krishna (1989) discusses the earthquakes and environmental problems of river valley projects in India and suggests the positive impacts of the projects in improving the life of people with minimal adverse impacts. Green et al (1983) discuss the accomplishments of the Columbia Basin Reservoir in the United States. They indicate that the annual benefits of the Columbia reservoir system are considerably greater than its annual cost and that it has greatly enhanced the quality of life in the Pacific Northwest. However there have also been some adverse impacts. A declining anadromous fishery resource is seen to have caused largely by dams that have inundated spawning beds and blocked or degraded migration routes. Spann (1983) presents environmental assessment and monitoring for the Arkansas River system in USA. He presents hydroelectric, navigational and other benefits of the multipurpose dam project and claims that the environment of the Arkansas river has improved to the point where it is enjoyed by the thousands of visitors with diverse interests each year. Cottereau and Plomb (1992) discuss positive aspects of the development of river Rhone in France. The multipurpose development of the river which commenced before the outbreak of the second world war has now been completed. They demonstrated that how the water resources of river Rhone are harnessed and converted into hydroelectric power which in turn provides the finance required for valley development and the execution of a truly integrated development project. It points the advantages of a series of low head projects to reconcile the various aims of the development programme, production of hydroelectric power, navigation, agricultural development with a minimum impact on the natural environment. There are many other studies appearing in the literature specifying both the positive and negative impacts of hydropower and multipurpose projects.

Some case studies of hydropower projects in India:

Silent Valley Project: A hydroelectric power project was planned in the State of Kerala with acute shortage of power resources. The project envisaged damming the Kuntipuzha river for hydropower generation and flooding part of the Silent Valley, India's only surviving virgin tropical rainforest. The project, though economically viable, was cancelled because of the environmental concern of the public. Khoshoo (1988) indicates that the Valley has a repository of biological diversity and that those who stalled the project were right because the

valley has contributed at least 20 genes for pest and disease control of rice thus serving the cause of humanity at large. Bhasin (1987) has however indicated that though the project has been shelved, the encroachment on the forest is going on and the beautiful forest is slowly disappearing. People argue that even in the absence of water development projects, the forests will go on disappearing because of the population pressures.

Tehri Hydro-power Complex: Tehri dam is a 260.5 metre high earth and rockfill dam being constructed downstream of Tehri town in the State of Uttar Pradesh to harness waters of river Bhagirathi for producing 2400 MW of hydropower. The project was approved by the Planning Commission of India and its site as well as design have the approval of many national and international experts. There is a strong public opinion against the project due to environmental concerns primarily on account of seismicity. In the Northern India, all the perennial rivers flow down from the Himalayas. Therefore all good dam sites are situated in the Himalayas, Tehri being one of them. As the Himalayas are a seismically active zone, doubts have been raised in some quarters about the safety of the dam. Although Tehri Hydro Development Corporation claimed that the design of the dam was based on latest techniques and had the approval of international experts from USA, Germany and USSR, many local people are not willing to take the risk and as such are against the continuation of the project. This is a case of risk assessment and uncertainty involved in the environmental assessment of water projects.

Sardar Sarovar Project: The Sardar Sarovar Project is a multipurpose river basin project for hydropower generation, irrigation and for providing drinking water supplies. The Sardar Sarovar dam is designed to diverget 9.5 million acreft of water from the Narmada river at Navagam and Kevadia into a canal and irrigation system. The dam is presently under construction. The project is one of the largest project undertaken, its impact extends over an immense area and may affect a very large number of people. Although the project has a bigger component of irrigation benefits than hydropower generation, some of the environmental issues of the project are relevant for large hydropower projects. The project is being financed partly from the credits and loans from the World Bank. A review of the project by Morse and Berger (1992) have specifically focussed on issues like rehabilitation of oustees and the potential damage to the environment and have suggested a halt to the construction activity. These issues of environmental impact of hydropower projects are important and will be discussed further in the next section.

Policy Issues:

It is now well recognized that the consequences of neglecting environmental factors from the early state of project identification to the later detailed work of project design and construction can be devastating both for the natural environment and for the human population it sustains. There is need to develop and use appropriate technologies and programmes which incorporate broad environmental and human concerns. These issues are analyzed below:

Large versus small system: High dams and large water development projects are presently under close and critical review of environmentalists. There is an extreme view that large projects are bad in themselves because of the possible adverse environmental impacts. This view is not justified. It is established that some large projects, as for example the Columbia and Arkansas river systems discussed earlier, have given immense benefits to the people. However, there are some attractive features of small and low head water power projects which must be considered in the overall planning of the projects. Fritz (1983) suggests that as oil and gas prices continue to rise, the capital costs of small hydropower are becoming increasingly attractive especially for remote areas and for developing countries. Goldsmith (1991) pleads for small hydro plants due to less problems of finance and environmental impacts. Riaz and Ali (1991) present the development of small hydro for northern Pakistan and the benefits derived for the inaccessible and remote areas. Small hydroplants are also favoured in India wherever feasible. Jai Krishna (1992) claims that tall dams have the advantage of spreading load over a large area of river bed to withstand earthquakes through slumping. He further opines that the cost of generating electricity per unit from a low dam (about one-eighth the height of a tall dam) is about five to six times that of a tall dam.

Technology level: Appropriate technology can be developed through research for hydropower development without impairing the environment at reasonable cost. The old principles and techniques of planning and design need modifications in view of the environmental factors and our experiences. Goldsmith (1992) indicates that modern system control technology can cope with rapid changes in the dispatching of relatively small block of power. Medium-sized run-of-river plants with only a small upstream pondage say in the range of 20-100 MW may be economically and environmentally more feasible and easier to develop. Xuemin (1922) discusses the ways by which

sedimentation problems of large hydroschemes in China have been monitored and dealt with. The use of pumped storage power plants requires lesser storage capacity of reservoirs and may be better wherever feasible for environmental considerations. Research and development efforts for their use are in progress in Japan and other countries.

Socio-economic factors: A large hydropower project may have both positive and negative socio-economic impacts. Displacement of people, loss of habitations and traditional livelihood pattern particularly in the tribal areas are serious adverse impacts. The problem of resettlement of oustees is becoming a matter of great concern. Razvan (1992) holds the view that the feasibility of a large dam takes into account compensation for damage and reasonable sums for the relocation of persons living in the impounded area. Moigne (1991) discusses various issues of resettlement problem and describes some good resettlement policies and plans based on the experience of more than 400 projects financed by the World Bank. He suggests that the key to successful resettlement lay in giving people enough productive resources to allow them to make a living after they had moved. Verghese (1990) suggests that those who are displaced need to be treated with the greatest consideration, sympathy and generosity. Solution to the resettlement and other social problems lies in the effective involvement of concerned people during the planning stage of the project. However, socio-economic environmental problems need to be considered in a larger perspective. Summing up the achievements of a large multipurpose water project in the United States, Evans (1977) writes: "The Tennessee Valley of today is a far different place than it was in 1933. A rampant river has been tamed, giving way to useful commercial navigation, to a beautiful utilitarian network of dams and reservoirs, to abundant low-cost electricity, poverty and despair have yielded to prosperity, eroded land to reforestation, recreation and agriculture. In the shadow of the environmental controversies that develop it today, to those who have never lived with kerosene lamps nor experienced the blight of massive unemployment, the Tennessee Valley Authority may still seem to be a senseless magnificent obsession."

Environmental Planning and Design: Although we have been developing hydropower projects around the world for more than a century, there is no lack of challenges for the future. We need to meet these challenges through technical, economic and environmental considerations in planning and design. A fundamental concern should be the

preservation and enhancement of compatible environmental diversity. As indicated by Moigne (1991) many of our problems would disappear if planners would give the same attention to designing appropriate solutions for resettling people (as an example) as they give to calculating concrete stresses, turbine sizes and engineering tolerance for machinery. Multiobjective approach may be used in the planning. In addition to maximizing economic benefits of the water projects we should also aim in minimizing adverse environmental impacts with the constraints that none of the serious impacts may be acceptable.

Conclusions:

Hydropower development is of vital importance to meet increasing energy demand of developing countries. Adverse environmental impacts can be eliminated or minimized by environmental planning and design, use of appropriate technology and by suitably considering socio-economic factors during the early stage of project identification.

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"Grizzly Powerhouse's Environmental Intake"

by
Blake D. Rothfuss[1], P.E., M.ASCE
William E. Zemke[2], P.E.

Abstract

Increased emphasis is being placed on the environmental impacts of new hydropower construction. Environmental factors are becoming an active part of the design process. In this project, fish protection and visual impacts became key factors influencing intake structure design, schedule and cost.

To minimize potential fish entrainment and turbine induced fish mortality, the California Department of Fish and Game (CDF&G) has a general policy of recommending that intake structures be equipped with continuously cleaned fish screens. In response to visual and public use concerns, the U.S. Forest Service (USFS) has taken an increased role in determining location, appearance and access for project features located on forest lands.

At the 20.2 MW Grizzly Powerhouse Project located in the Plumas National Forest in Northern California, the intake structure was designed to minimize cost, meet engineering and operating needs, prevent fish entrainment, present a visually acceptable appearance, minimize "re-design" risks, and meet schedule requirements. The intake structure is located at the northern shore of Lower Bucks Lake with an invert elevation of 4983 and approximately 40 feet of head. The intake is located approximately 200 feet from an existing dam and spillway. The intake structure features a stainless steel fish screen with ¼-inch slotted openings, an approach velocity of 0.5-feet-per-second, and an automatic trashrake to clean the fishscreen.

Background

The Grizzly Powerhouse Project (Grizzly) is an addition to the existing Bucks Creek Project (FERC No. 619) located east of Oroville, California in the Sierra Nevada Mountains. In 1981, Pacific Gas and Electric (PG&E) filed an application with the Federal Energy Regulatory Commission (FERC) for amendment of license to

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[1]Lead Civil Engineer, Pacific Gas and Electric Company, San Francisco, California
[2]License Coordinator, Pacific Gas and Electric Company, San Francisco, California

construct Grizzly to make use of the existing 700 feet of head between Lower Bucks Lake and Grizzly Forebay. Lower Bucks Lake diverts water from Bucks Creek through an existing tunnel and into Grizzly Forebay, which is the forebay for the existing Bucks Creek Powerhouse. In 1981 preliminary design of the Grizzly intake structure began, based on a low-level, unscreened intake, similar to existing intake towers at Lower Bucks Lake and Grizzly Forebay.

Grizzly is located entirely on USFS lands which contain fishery resources important to CDF&G. These agencies became important participants in the licensing and design process. A memorandum of understanding was reached with the USFS concerning project elements that were important to that agency. Discussions were also held with CDF&G, but a final agreement was not reached. Final agency comments were submitted to the FERC. One of the recommendations of CDF&G was that the Grizzly intake include a fish screen.

The FERC issued the Grizzly license amendment in 1988. The license amendment included numerous environmental mitigation requirements, including screening the intake as recommended by CDF&G. Upon appeal, the FERC modified the screening requirement to require that a fishery study plan be developed and that post-operational studies be conducted for at least two years to investigate the benefits of screening, with the potential for future installation of a screen on the intake if the studies determined one to be necessary. PG&E decided to proceed with the simultaneous design of both the original low-level, unscreened intake and a high-level, screened intake as the potential issue of fish entrainment was considered more closely.

In 1989, PG&E and the City of Santa Clara (City), a municipal electric utility near San Francisco, entered into a cooperative development agreement to construct Grizzly. Under this arrangement PG&E is the Project Manager, Project Engineer, and Construction Manager, and will operate Grizzly for the City for an initial two year period. The City will own Grizzly and receive the power it generates. PG&E has an option to purchase Grizzly from the City in the future. PG&E and the City are joint licensees under the FERC license. PG&E is constructing the majority of Grizzly under a single, prime construction contract with Guy F. Atkinson Construction Company of California.

Project Description

The Grizzly intake structure is located at Lower Bucks Lake. An eleven foot diameter TBM-driven pressure tunnel will lead from the intake structure at Lower Bucks Lake to a surge tank and penstock. A steel penstock approximately 4,900 feet long leads from the end of the tunnel to the powerhouse located on the shore of Grizzly Forebay. A turbine with pressure regulating valve (PRV), generator, and turbine shutoff valve is located in the powerhouse and will discharge into Grizzly Forebay. The PRV conduit also serves as a powerhouse bypass to maintain flow to Grizzly Forebay when the Grizzly turbine-generator unit is out of service. Grizzly Forebay supplies water to PG&E's Bucks Creek Powerhouse.

The Grizzly Powerhouse Project includes the following general features:

Intake: Submerged horizontal funnel type, reinforced concrete, with fish screen, trashrake, hydraulic fixed-wheel gate, and bulkhead gate.
Tunnel: 12,603' long, excavated by an 11 ft. diameter tunnel boring machine (TBM) and conventional methods.
Surge Chamber: 212' tall by 10' to 15' diameter (finish) reinforced concrete-lined vertical shaft with steel-lined orifice.
Penstock: 4,900' long by 96" to 56" diameter, steel, buried for most of the length, supported by ring girders and piers in above ground sections.
Powerhouse: 65' x 54' reinforced concrete structure, designed for unattended operation.
Turbine Shutoff Valve (TSV): Butterfly type.
Turbine: Vertical shaft, Francis type, 26,400 HP.
Generator: Vertical shaft, synchronous, 22,000 kVA, 0.90 power factor, 6.9 kV, 60 hertz, 3 phase, 450 rpm.
Pressure Regulator/Bypass Valve (PRV) System: Howell-Bunger type and rupture discs.
PRV Guard Valve: Ball type.
High Voltage Equipment: Step-up transformer and power circuit breaker.
Switchyard: Approximately 14,500 sq. feet, paved with asphalt concrete.
Transmission Line: 115 kV, single circuit, three conductors, H-frame wood pole line 3.4 miles long.
Distribution Line: 12 kV, single circuit, three conductors, wood poles, mostly underhung on 115 kV line, 1.4 miles long.
Telecommunication Facilities: Fiber optics, copper cable, and microwave.
Roads: 25 miles of new, improved and restored roads.
Recreation Facilities: 7 camp sites, 30 picnic sites, one boat ramp, and related facilities.

Intake Design Evolution

Grizzly's intake structure is screened to prevent fish entrainment. The nearly completed structure is the balanced product of hydroelectric functional requirements, agency requirements, project schedule and economics, and environmental concerns. The screened intake will divert generating waters to Grizzly powerhouse in an economic and environmentally sound manner.

The intake initially designed by PG&E was an unscreened low-level intake, drawing water from the lower strata of Lower Bucks Lake. The tower design was similar to existing intake towers present at Bucks Lake, Lower Bucks Lake, and Grizzly Forebay. Because of the depth of the intake (over 60 feet), fish entrainment was believed not to be an issue, and the intake was unscreened. A trashrack with a 2" clear opening and a velocity between the bars of 2 ft./sec. would prevent debris from entering the waterway. In order to minimize excavation costs, the tower's foundation would be located inside the reservoir away from the shoreline. Access would be provided via a binwall causeway running along the shoreline out to the tower. A manually operated fixed wheel gate and water level metering/ telemetry would be the only equipment housed in the gatehouse atop the tower. Figure 1 profiles the original unscreened low-level intake.

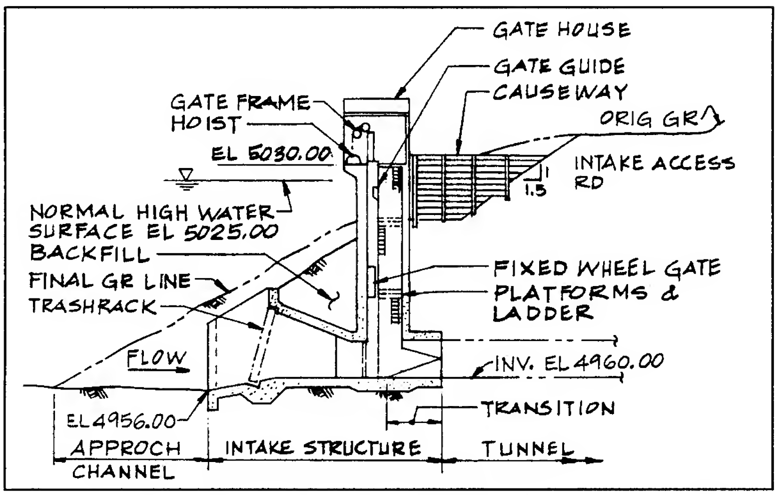


Figure 1: Unscreened Low-Level Intake Profile

The final design schedule did not permit PG&E extensive time to attempt to negotiate agency support for the partially complete unscreened low-level intake design, so the decision was made to start design of a screened intake structure during the negotiations. PG&E began an intensive design review of the original unscreened low-level intake, and contracted with Bechtel Corporation to simultaneously design a screened high-level intake.

Screened High-level Intake

The screened high-level intake was a substantially larger structure than the unscreened low-level design to produce a slower approach velocity and accommodate a trashrake. The elevation of the top of the screen was set as high as possible while still preventing the formation of vortices when operating at the minimum water level. Decreasing the approach velocity from 2-feet-per-second to 0.5 feet-per-second quadrupled the required screen area. In order to accommodate a 790-square-foot fish screen, the structure was widened and lengthened to improve hydraulic performance. The structure is 68-feet long by 47-feet wide and 57-feet from invert to operating deck. A 26-feet long by 27-feet wide gatehouse 22-feet tall sits atop the structure. Figures 2 & 3 present the screened high-level intake plan and profile; Figure 5 presents a photograph of the intake. The significant features of the screened high-level intake are summarized below.

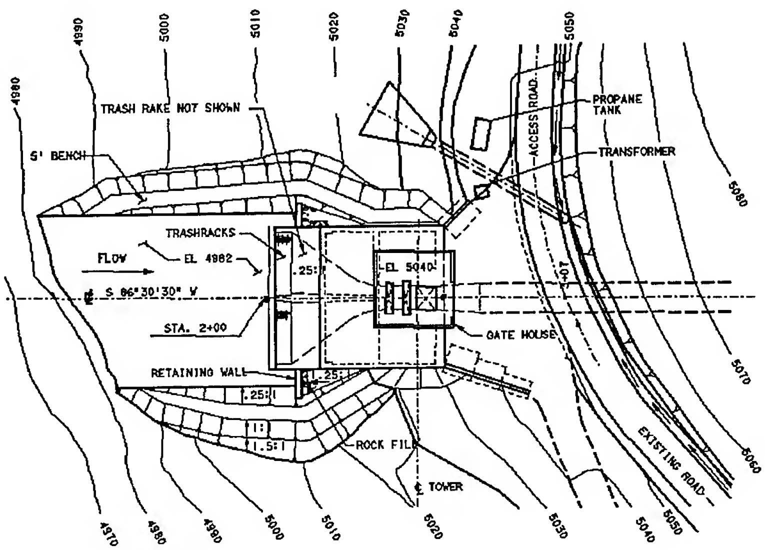


Figure 2: Screened High-Level Intake Plan

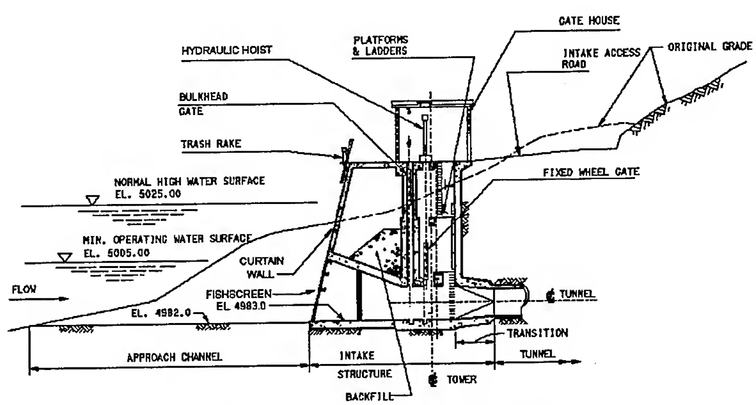


Figure 3: Screened High-Level Intake Profile

Fishscreen: The fishscreen is a prefabricated wedge-wire assembly manufactured by Bixby-Zimmer (BeeZee). The vertical Grizzly-rod stainless steel bars taper in width from $^3/_{16}$" wide (upstream) to $^1/_8$" wide (downstream) and are $^{11}/_{32}$" deep. The 5-degree taper prevents material from binding between the bars since the narrowest open cross-section is on the upstream face of the screen. There is a $^1/_4$" gap between

the bars at the face of the screen. Figure 4 presents a typical BeeZee Grizzly-Rod configuration and Figure 6 shows the installed fishscreen. The tapered bars are connected to a horizontal stainless steel $^1/_4$" by $^3/_4$" tie rod which is then fastened to a structural steel panel. There are eight 5'-10" tall by 20'-6" long panels on the front face of the intake structure; four each side of centerline. The top two panels are fastened to the outside of the structure enabling a diver to unfasten and remove a panel without having to work from the inside of the structure.

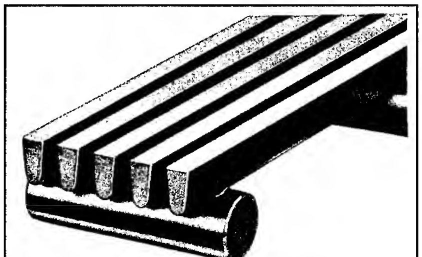


Figure 4: BeeZee typical Grizzly-rod configuration. Note: Grizzly's screen uses rectangular tie rods instead of the round tie rods shown.

Trashrake: To clean the bottom of the fish screen to a depth of approximately 60 feet below the operating deck, PG&E specified a double boom hydraulically operated trashrake. The double boom assembly minimizes the size and weight of the trashrake and presents a low-profile visually-acceptable cleaning mechanism. Typically, these trashrakes function as trashrack cleaners. However, in order to operate as a fish screen cleaner, it was necessary to redesign the raking head to incorporate a Teflon wiper. As with conventional trashracks, debris wiped off the fish screen will be brought to main deck level for disposal.

Atlas Polar's Hercules Hydrorake Model DT8300 travels back and forth across the face of the intake to remove debris from the screen. The double boom traps debris between the wiper blade and the curtain wall. The debris is deposited on the main deck where maintenance crews will collect and dispose of it. The main deck is designed to accommodate an Atlas Polar trash conveyor if trash handling costs warrant.

The Atlas Polar trashrake was selected because of its low profile superstructure and its operating characteristics. The trashrake will operate within temperature extremes dipping below zero degrees Fahrenheit to above 112 degrees Fahrenheit, by using heaters and food grade hydraulic fluid. Use of food grade oil is intended to reduce fishery impacts in the event of a hydraulic leak. The rake can be programmed to operate continuously or intermittently and can also be programmed to operate based on head loss across the screen as measured by a differential level switch connected to float wells. Similar Atlas Polar trashrake systems run quietly and efficiently at several other PG&E hydroelectric intakes.

Curtain Wall: One of the major differences between the unscreened low-level intake and the screened high-level intake is a curtain wall above the screen. The addition of a trashrake to the screened high-level intake design necessitated a means to dispose of any debris the rake retrieved. This requires transporting the debris from the screen to

the main deck of the intake structure. The curtain wall links the screen and the main deck. PG&E considered using structural steel with epoxy coating, stainless steel, or concrete for the curtain wall but selected reinforced concrete because of its toughness and durability. To minimize noise when the trash rake wiper moves across the concrete and to prevent excessive wear of the trashrake's Teflon wiper, the concrete face of the intake structure is coated with Carboline 163-2 and topcoated with Carboline 891, both100-percent solids epoxies.

Access Road: The unscreened low-level intake access road was originally designed to be constructed along the shoreline of Lower Bucks Lake using binwalls. The 300' roadway would be visible from much of the perimeter of the lake and would interrupt the natural transition of forest to lake. When the screened high-level intake was designed as a shoreline intake, the need for a long and unsightly access road was eliminated. Instead, an existing Forest Service access road was lowered and improved to provide access to the intake.

Gatehouse: The gatehouse contains a fixed wheel gate hydraulic operating system, bulkhead gate wire rope hoist, float well and stilling basin monitoring equipment, batteries, and telemetry equipment. The gatehouse architecture visually complements the surrounding forest and shoreline. The walls of the gatehouse are constructed from light brown split block masonry units selected to visually match the weathered diorite rock shoreline. The insulated steel roof is painted a forest green to blend with the forest behind the intake. Table 1 summarizes the features of the two intakes.

Table 1: Intake Comparison Summary **Description** (All Elevations on USC&GS Datum)	**Unscreened Low-level Intake**	**Screened High-level Intake**
Main Deck Elevation	5030.00	5040.00
Normal Maximum Operating Water Surface Elev.	5025.00	5025.00
Minimum Operating Water Surface Elevation	4987.50	5005.00
Minimum Construction Water Surface Elevation	4987.50	4987.50
Invert Elevation	4960.00	4983.00
Foundation Elevation	4956.00	4980.00
Design Flow, maximum, cfs	395	395
Trashrack Approach Velocity, fps Fishscreen Approach Velocity, fps Actual Approach Velocity, at design flow, fps	2.0 Not Applicable Not Applicable	Not Applicable 0.50 0.44
Required Screen Area, square feet Provided Screen Area, square feet	198 Not Applicable	790 907
Trashrake Reach, maximum, feet	Not Applicable	61
Access Road Length, feet	300	30
Intake Dimensions, Reinforced Concrete Gatehouse Dimensions, Split Block Masonry	76' long x 20' wide x 70' tall 21' long by 20' wide by 19.5' tall	68' long x 47' wide x 57' tall 26' long by 27' wide by 22' tall
Amount of Concrete Visable from Lakeside	100 SF	715 SF
Fixed Wheel Gate Operation	Manual	Automatic

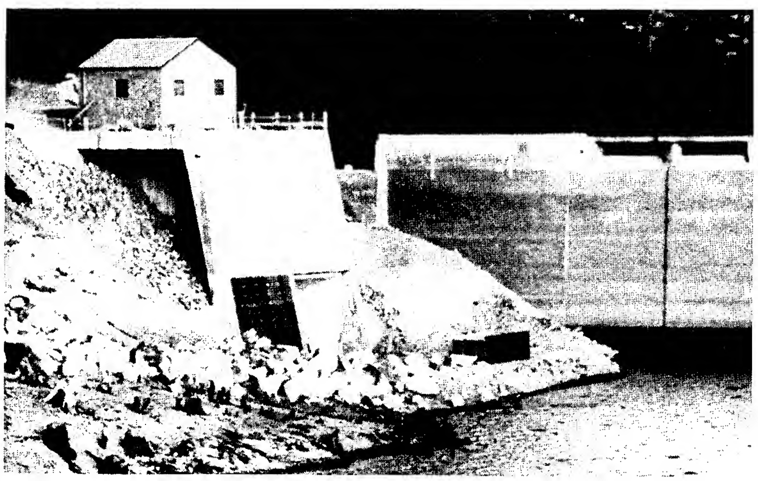
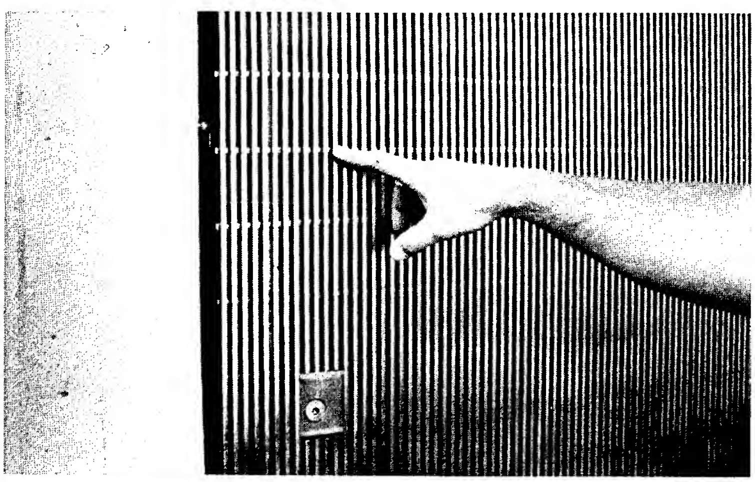


Figure 5 (above): Grizzly's Screened High-Level Intake Structure with Lower Bucks Lake Dam in the background; **Figure 6 (below):** A close-up view of the fishscreen



Decision Analysis

PG&E conducted preliminary fishery studies and held discussions with CDF&G and other fishery agencies to assess the potential need for a fishscreen. During the discussions with these agencies, several alternative options were considered ranging from prevention of fish entrainment to mitigation. The fishery study results, while

supporting expectations that fish entrainment would not be a problem for an unscreened low-level intake, were not sufficiently conclusive to eliminate the risk of future screening requirements. Retrofitting the unscreened low-level intake structure to include a fish screen and trashrake after construction and initial operation would be a major expense.

Although the question of whether to screen or not to screen was the principal issue, other environmental issues including approach velocity and aesthetics were also considered. CDF&G standards for fish screens include an approach velocity of 0.33 feet per second. Discussions resulted in CDF&G's agreement that an approach velocity of 0.5 feet per second was acceptable.

The decision to proceed with the final design and construction of the screened high-level intake was based on environmental, operational, and design considerations in addition to construction cost. These considerations led PG&E to decide to proceed with the screened high-level intake design, even though it was the more expensive alternative. Proceeding with the screened high-level intake would eliminate the need for performing post-operative fishery studies and would eliminate the risk of being required to provide potentially costly mitigation if studies showed entrainment to be a problem. Additionally, PG&E would not have to redesign the unscreened low-level intake so that it could be retrofitted with a fish screen and trashrake if required in the future.

When the conceptual design of the screened high-level intake was completed, construction cost estimates were prepared for both the screened high-level and unscreened low-level intake designs. PG&E evaluated the cost items which were affected by the location, size, and layout of the alternative designs. They were site work, cofferdam, dewatering, excavation, concrete, fishscreen & frame, trashrake, and portal & tunnel improvements.

Sitework, Excavation, and Concrete: The size of the excavation, ground support, drainage, and structure differ substantially between the unscreened low-level and the screened high-level intake designs.

Cofferdam and Dewatering: Construction of the unscreened low-level intake would require a sheet pile cofferdam approximately forty feet high with dewatering pumps to keep the excavation workable. The screened high-level intake foundation would be twenty-four feet higher and closer to shore than the unscreened low-level intake, and a six-foot high earth berm would be used as a cofferdam.

Portal & Tunnel Adjustment: Generally, the degree of rock weathering at the intake site decreases with depth. Raising and moving the intake toward the shore increases the amount of weathered material which must be supported during construction. An adjustment was made for additional rock bolts,

shotcrete, and other ground support. If the intake were founded at the lower level, the tunnel would be forty feet longer than for the high-level intake. An adjustment was made for increased tunneling costs.

Fixed Wheel Gate Adjustment: The gate stems for the unscreened low-level intake would be thirteen feet longer than for the screened high-level intake. Accordingly, additional stems, guides, and storage would be required to make the fixed wheel gate operable.

Conclusion

The design criteria for the Grizzly Powerhouse Project intake structure, while considering varied engineering and operating needs, was also strongly influenced by environmental, regulatory, and schedule considerations. The resulting final design was a compromise between environmental considerations and the constraints of engineering and operating needs, project schedule, project economics, and risk management. Although fish screens are not necessarily the standard for the future, the need to consider this and other environmental issues is clear. The challenge for the industry will be to recognize the dynamics of these constraints early on and work to meet increasingly diverse needs in cost effective ways.

Acknowledgments

The authors with to thank Mr. David Moller of Pacific Gas and Electric Company and Mr. John Schwartz of the City of Santa Clara for their guidance during the development of this paper. We also wish to thank Messrs. David Cherry and John Schmiedel of Bechtel Corporation for their contributions to the Grizzly project.

COST-EFFECTIVE SOLUTIONS TO FISHWAY DESIGN

by Peter J. Christensen[1]

Abstract

An increasingly common requirement placed on hydroelectric power generation in the United States is the installation and operation of fish passage facilities. Historically, this entailed installation of the traditional fish ladder or elevator type upstream passage facility. Recent requirements also include the construction of downstream passage facilities to mitigate blockage or delay of downstream migration and entrainment of fish in the project turbines. The cost of constructing these facilities, and the lost revenue associated with their operation, can represent a substantial financial burden. Preliminary design investigations and creative design techniques can sometimes reduce these costs without detrimentally affecting the function of the fish passage facilities.

This paper presents some of the design techniques which have been used to reduce fishway size and the impact of fishway flow requirements on hydroelectric generation. Examples are presented of some existing upstream fishways which have used creative solutions to minimize the impact of fishway flows on station generation. The emphasis of the paper, however, is a more detailed presentation of methods currently being used to reduce the cost and impact of downstream fish passage facilities. This paper presents a comparison of two downstream fish passage facilities, designed by Kleinschmidt Associates, which were constructed in Maine during the summer of 1992.

Background

For each fish passage facility, upstream or downstream, there is generally an associated attraction flow dictated by the governmental agencies responsible for reviewing and accepting the design. The purpose of the attraction flow is to create a suitable flow field in the vicinity of the facility's fish entrance designed to attract

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[1] Civil Engineer and Fisheries Biologist, Kleinschmidt Associates, 75 Main Street, Pittsfield, Maine 04967

fish into the system. Although specific site requirements may vary, typical values required in the eastern United States are 2 to 3% of station hydraulic capacity for an upstream fishway and 2% for a downstream facility.

A common misconception is that the attraction flow represents the flow which must be passed through the entire system. The attraction flow is actually only required at the fish entrance end; this is the flow exiting the bottom of an upstream fishway or the flow entering the top of a downstream fishway. The flow which is required to effectively move fish through the fishway once they have entered it is defined as the transport flow.

The transport flow requirement of an upstream passage fishway is a function of many factors, including: the type and size of fishway being employed, the species and anticipated quantity of fish being targeted for passage, the hydraulic velocities within the system, required pool volumes, and energy dissipation per volume within the system. A presentation of how to determine this flow requirement is beyond the scope of this paper. Downstream fishways are generally much simpler in design in that once the fish have entered the system it is only required that they be "sluiced" without injury to the tailrace. An accepted minimum transport flow for many downstream fishways in the eastern U.S. is 0.57 m^3/s (20 cfs). In either case, upstream or downstream passage, the required transport flow is often considerably less than the required attraction flow.

The difference between the attraction flow and the transport flow represents a flow of water which does not necessarily need to be "wasted" down the entire fishway. The transport flow can be supplemented at the fish entrance end of the system with water which has previously been, or will subsequently be, used by other project facilities. In this way, the supplemental attraction flow can be "recovered" and used for more than one purpose, thus reducing the loss to project revenues.

Upstream Fishway Design

Methods of providing upstream migrating fish safe passage around hydroelectric projects include: various types of fish ladders, fish elevators, fish locks, navigation locks, and trap-and-truck facilities. Many of these facilities have incorporated the design concept of supplementing the fishway transport flow at the downstream end to obtain the proper attraction flow exiting the system. Different methods have been used successfully to supply this supplemental flow. Some of these designs take advantage of this flow by passing it through the station turbines prior to its introduction into the downstream end of the fishway.

Attraction Water Pipe:

The simplest method of supplying the additional attraction flow is directly from the headpond via a secondary attraction water pipe. Although this method does not generally make any additional use of this water, it does reduce the size of the fishway itself and offers the benefit of flow regulation by simple valve adjustments. This ability can be advantageous since reductions in the attraction flow may be justified during periods of low flow when the station is running below its hydraulic capacity. Future permanent reduction of the flow requirement may also be justified if the fishway's effectiveness at the reduced flow can be documented. The ability to regulate this supplemental flow can also be advantageous if variations in the headpond cause the transport flow to fluctuate.

Supplemental Pumping:

A method of recovering some of the energy potential associated with the supplemental attraction flow is to pump the additional flow requirement directly out of the tailrace into the downstream end of the fishway. This pumping can be done with relatively inexpensive submersible mixer pumps, with low energy demand, since it is being done against a very small head differential. For example, if the supplemental flow is pumped against 0.3 m (1 ft) of head at a 50% pump efficiency, the total loss to net station energy from operating the fishway is equal to the transport flow at full station net head plus the supplemental attraction flow at 0.6 m (2 ft) of head. Even at relatively low-head stations, this can represent a significant reduction in generation loss as compared to passing the entire attraction flow down the fishway and/or secondary attraction water pipe. In many instances this reduction in loss can pay for the additional equipment and construction costs associated with pumping in the first couple of years of operation.

According to a telephone conversation with Ben Rizzo, a hydraulic engineer with the U.S. Fish and Wildlife Service, the largest example of supplemental flow pumping on the east coast is presently at the West Enfield Dam on the Penobscot River in Maine where attraction flow requirements for upstream passage can be as high as 11.3 m^3/s (400 cfs).

Minimum Flow Unit:

At many sites there is a minimum flow requirement in the river downstream of the station. This minimum flow is often below the minimum turn-down capabilities of the existing turbines. The addition of a minimum flow unit, to operate during periods of low flow and to supplement station capacity during periods of high flow, can often be economically justified. Designing this unit to also supply the supplemental attraction flow presents an excellent use of water.

At the Wilder Project on the Connecticut River, a creative design approach was used by the New England Power design team (Doret, 1987). In conjunction with the construction of an upstream fishway, the powerhouse was expanded and a new 3.2-MW unit was added. This unit not only supplies the supplemental attraction flow to the fishway, it also is designed to operate efficiently at the 19.1 m^3/s (675 cfs) minimum river flow requirement. The existing two 17-MW Kaplan units operated at very low efficiency and experienced substantial cavitation when run at the minimum river flow. This new unit has a gated tailrace so that its tailwater level can be controlled. The downstream end of the fishway consists of a collection gallery which extends across the tailrace of this new unit. The floor of the collection gallery is made up of grating, allowing some of the tailwater from the new unit to pass up into the fishway and combine with the transport flow.

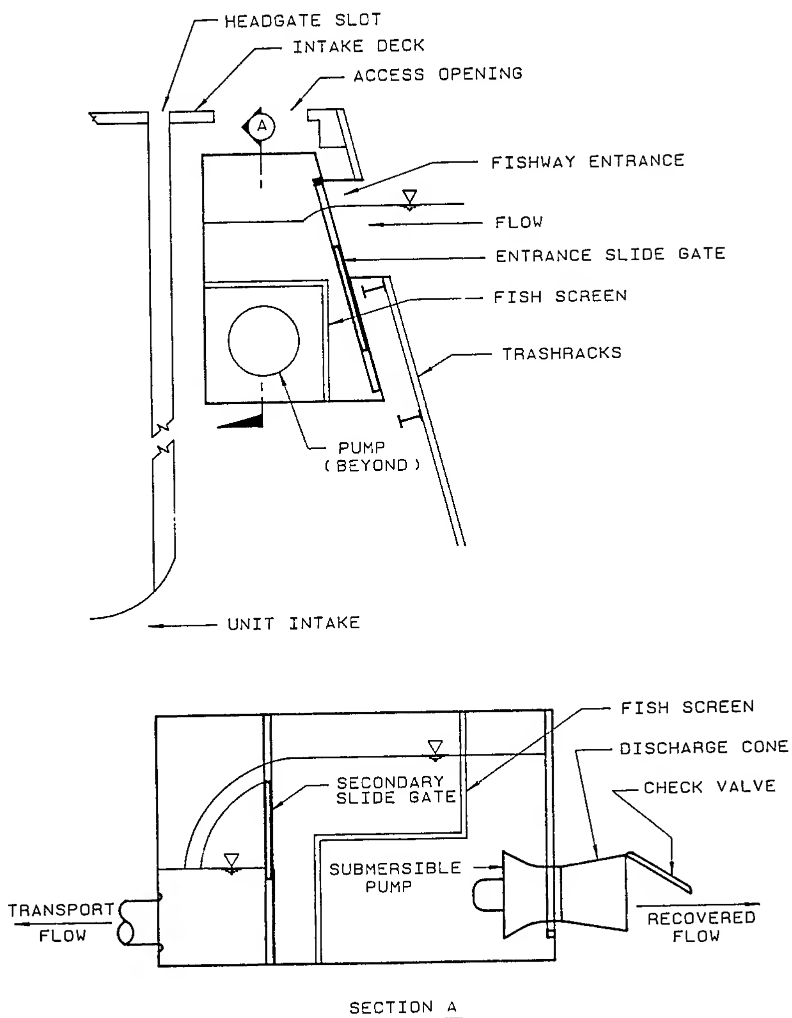
Downstream Fishway Design

A common design of downstream fishways in the Eastern U.S. consists of a gated entrance, collection chamber, and transport pipe. A fish screening or guiding device is also commonly provided to minimize turbine entrainment and maximize fish bypass efficiency. The location of the fish entrance should be placed so as to take advantage of the attraction potential of the station flow. The most effective means of accomplishing this is to place the entrance within the station intake behind the trashracks (see Figure 1). A small section of the trashracks is cut out and removed to allow unobstructed flow and fish travel through the entrance into the collection chamber. Flow into the collection chamber is regulated by a downward-opening entrance slide gate or stop logs. After entering the collection chamber, the fish are diverted into a transport pipe which may deliver them directly to the tailrace, to an appropriate sluiceway, or to a fish sorting and collection facility.

Pump-Back Concept:

Once the attraction water and fish have entered the collection chamber, the quantity of flow in excess of the transport flow can be removed and returned to the headpond, allowing it to be used for power generation. This can be accomplished with a pump, or pumps, attached to the collection chamber, as shown in Figure 1. Since the head loss involved in passing water over the entrance weir gate is small, this recovery of flow can be accomplished with relatively inexpensive, submersible, mixer-type pumps. These pumps are generally mounted on rails for easy installation and removal should maintenance be required. At net pumping heads of 0.46 to 0.76 m (1.5 to 2.5 ft), pumps capable of pumping 1.1 to 1.4 m^3/s (40 to 50 cfs) are readily available.

Adding recovery pumps to the collection chamber design does entail some additional equipment and construction costs when compared to a simpler design



DOWNSTREAM FISHWAY COLLECTION CHAMBER

FIGURE 1

which diverts the entire attraction flow into the transport pipe. However, there are two sources of savings involved with this type of recovery system which can offset this cost. There is the long-term savings associated with recovering some of the energy which would be lost if the entire attraction water flow was "wasted" down the fishway. There is often also a direct cost savings associated with the construction of the transport portion of the fishway since the flow in this portion is reduced.

Common Flow for Upstream and Downstream Attraction:

If a site has both an upstream and downstream fishway which need to operate during the same time periods, then the same water can be used to serve as attraction water for both. This is accomplished by separating the excess attraction water at the collection chamber of the downstream fishway and piping it to the downstream end of the upstream fishway. This can be an attractive alternative at very low-head stations where the energy recovery realized from passing this flow through the station units does not justify the energy requirement or capital cost associated with pumping it at both locations.

Recent Case Studies of Downstream Fishway Design

The following two case studies describe downstream fishways, designed by Kleinschmidt Associates, which were constructed and placed in operation during 1992. These fishways are located on major rivers in Maine and were primarily designed to pass migrating Atlantic salmon. In both cases a preliminary evaluation was performed to determine the economic benefit of recovering some of the energy loss associated with the attraction flow requirement. Recovery proved to be beneficial in one of the two cases. Numerous physical, operational, hydraulic, and economic site conditions at the second project combined to make the pump-back recovery approach too costly. A comparison of these sites reveals a useful list of items to keep in mind when considering this type of design at other projects.

Case Study #1:

This fishway is located at a 9-MW station in Maine. The station's hydraulic capacity is 85 m^3/s (3000 cfs) at a net head of approximately 13.5 m (44 ft). The owner was required to install a downstream fishway with an attraction flow of 1.7 m^3/s (60 cfs) or 2% of station capacity. However, the transport flow was allowed to be as low as 0.57 m^3/s (20 cfs). The fishway is to operate each year from April 1 to November 30.

The design of the collection chamber at this fishway is similar to that shown in Figure 1. After the attraction flow enters at the entrance slide gate it is split into two components, 1.1 m^3/s (40 cfs) is pumped back into the headpond

behind the trashracks and 0.6 m^3/s (20 cfs) is diverted to the transport pipe. Once the fish have entered the transport pipe, they are delivered directly to the tailrace.

Additional construction costs incurred by the project, when compared to a simple design which passes the entire 1.7 m^3/s, were associated with the following items: The pump, discharge cone, check valve, and control panel cost \$25,000. The secondary slide gate, required to control internal water levels and thus pumping flow rates, cost \$7,000. The fish screening and support framing cost approximately \$1,000. Additional steel fabrication, construction, and installation costs were approximately \$5,000. Therefore, the total increase in initial investment was approximately \$38,000.

The operational requirements of the fishway are that it be in full operation for 244 days a year. The pump manufacturer's efficiency information cites the energy requirement to pump 1.1 m^3/s (40 cfs) against the required 0.46 m (1.5 ft) of net head differential as 11 kw. If it is conservatively assumed that the continuous requirement would be 15 kw, the annual energy requirement to run the pumps is 88 MWH. Having an additional 1.1 m^3/s available for station generation at a normal net head of 13.5 m (44 ft) represents an increased station production of 112 kw. Analysis of the flow-duration data for the site revealed that on the average there would be 190 days per year between April 1 and November 30 during which the river flow would be below the station capacity and the station could therefore make use of the recovered flow. This results in an average annual recovered energy generation of 510 MWH. Assuming a value of \$60/MWH, this recovered energy is worth \$30,600/year. The cost of running the pumps is \$5,300/year. This represents a net annual benefit of \$25,300.

As this analysis shows, recovering the excess attraction water pays for the additional costs of construction and equipment within a relatively short period of operation. Therefore, final design and construction of this facility included a recovery pump as shown in Figure 1.

Case Study #2:

In this second case it was required that a downstream fishway be constructed at an 18-MW station in Maine. The station has four units with a total hydraulic capacity of 198 m^3/s (7000 cfs) at an average net head of 11.6 m (38 ft). The fishway was required to have two entrances with a total attraction flow of 4.0 m^3/s (140 cfs) or 2% of capacity. The transport flow was only required to be 0.57 m^3/s (20 cfs). The fishway must operate during the months of April, May, June, and November every year.

An analysis was performed to determine the economic feasibility of recovering 3.4 m^3/s (120 cfs) from the attraction flow and returning it to the headpond for power generation. Although this would intuitively appear to be an

excellent opportunity for attraction flow recovery, this particular site presented many obstacles to installing a pump-back system.

Non-standard generation characteristics of the site dictated that the energy to run the pumps could not be supplied by the station. The station is supplied by the local utility with power for station service, however, this hook-up is not capable of handling the load which would be required to run the recovery pumps. Due to the remote location, the cost to upgrade the transmission line was estimated by the local utility to be approximately $100,000.

The physical layout of the gatehouse prevented pumps from being placed on the sides of the collection chambers as shown in Figure 1. Each of the station's four units has an individual trashrack and intake separated from the others by concrete walls 1.8 m (6 ft) thick. Behind the trashracks, each intake is further divided by a concrete wall into two bays each 3.7 m (12 ft) wide. Due to the spatial limitation of 3.7 m, the only way to install pumps on the collection chambers would be to have them mounted on the bottom of the chambers. This design was unacceptable to the station's operations personnel since the pumps would be accessible only by divers and would present maintenance difficulties.

A design was then investigated which involved excavating and installing a pump chamber outside the gatehouse. The only location available consisted of an earth embankment which was part of the project's water-retaining structures. Buried within this embankment was a concrete core wall which could not be disturbed. The location of this core wall greatly restricted the area available for installation of the pumping chamber, further complicating design and increasing costs of excavation and construction.

The additional estimated construction costs associated with the pump chamber design described were significant. The pump and related equipment cost was $46,000. The additional excavation and concrete demolition was $28,000. The concrete pumping chamber was $60,000. The pumping chamber accessories including fish screening, pump manifold, additional gate, work platform, and chamber cover were $41,000. Including the $100,000 quoted by the utility, the additional estimated construction cost for recovery pumping totaled $275,000.

The power needed to pump the 3.4 m^3/s (120 cfs) though all the required piping back to the headpond was estimated to be 100 kw. Due to the high flow rates in the river during April and May, of the 121 days that the fishway is scheduled to be in operation per year, only 66 days in an average year would experience flows below station capacity. Operating the pump for the average 66 days per year would cost an estimated $9,500/year. The additional station generation realized by passing the recovered flow through the units for the same 66 days would be worth approximately $14,700/year. The average net benefit, therefore, is estimated to be $5,200/year.

Operating the pump for the entire 121 days and only realizing benefits for 66 days would obviously not be cost effective. Since the pump would be off-line during a substantial portion of the required fishway operation period, and due to the sensitive nature of this location from a fish restoration perspective, the agencies involved required that the complete system be capable of passing the entire 4.0 m^3/s (140 cfs) to the tailrace in the event that the pumps are turned off or break down. This requirement negated any savings which may have otherwise been realized in the material and construction costs of the remainder of the fishway associated with passing reduced flow.

The $5,200 average annual benefit was obviously unattractive when compared to the estimated $275,000 additional construction cost. Therefore, the fishway was designed and built to pass the entire 4.0 m^3/s attraction flow to the tailrace without provisions for flow recovery pumping.

Conclusion

The concept of attraction flow recovery can often be used to reduce the economic impact of fishway operation for both upstream and downstream fishways. In certain instances, as was experienced in Case Study #2, existing site conditions may combine to make flow recovery unattractive from an economic standpoint. However, at many sites a careful analysis of site conditions will show some amount of flow recovery is economically justified. Site conditions which should be considered in any analysis include: existing station layout, spatial restrictions, biological requirements and sensitivity, value and cost of power, and the magnitude of river flows during required fishway operation periods. The design engineers involved in any fishway design should work closely with the biologists and resource agencies to produce a product which not only passes fish safely and effectively, but also reduces to a minimum the negative effects that the fishway operation will have on energy production.

Reference

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FISH PASSAGE/PROTECTION COSTS AT HYDROELECTRIC PROJECTS

J. E. Francfort, B. N. Rinehart, G. L. Sommers[1]

ABSTRACT

The U.S. Department of Energy's Hydropower Program is engaged in a multi-year study of the costs and benefits of environmental mitigation measures at hydroelectric power plants. The initial report (Volume I. Current Practices for Instream Flow Needs, Dissolved Oxygen, and Fish Passage - December 1991) reviewed and surveyed the status of mitigation methods for fish passage, instream flows, and water quality. Information on mitigation practices at non-federal hydroelectric projects was obtained from Federal Energy Regulatory Commission databases, provided by hydroelectric developers, and provided by state resource agencies involved in hydroelectric regulation. The types of mitigation costs incurred by the hydroelectric developers and examined include: capital, study, operations and maintenance, annual reporting, and lost generation costs. The costs are reported by capacity categories.

While Volume I was a "broad brush" study, the Volume II report focuses in detail on the costs and benefits of fish passage and protection measures. This involves an in-dept analysis of projects reporting upstream and downstream fish passage and protection mitigation. Case studies and information from developers are utilized to acquire detailed information for all incurred costs. This paper will examine the costs and frequencies of fish passage/protection environmental mitigation.

BACKGROUND

Environmental mitigation requirements at hydroelectric projects are intended to minimize any adverse environmental impacts that may be caused by the development and operation of a hydroelectric project. Mitigation measures always

[1] Members of the Hydropower Program at the Department of Energy's Idaho National Engineering Laboratory. Employed by: EG&G Idaho, P.O. Box 1625 M.S. 3526, Idaho Falls, Idaho, 83415-3526. (208) 526-6787.

include an economic cost, be it the purchase of land for habitat, the construction of capital structures such as ladders or screens, changes in operating mode from peaking to run-of-river, or bypass flow requirements which result in loss generation being augmented by other, more expensive fuels. While the hydroelectric developer, be it an entrepreneur, utility, or municipality, directly pays the costs in loss revenue, it is society that ultimately incurs these costs in the form of higher electric power expenditures. While not suggesting that mitigation is unwarranted, the costs of mitigation should be identified and considered as a tool to value the identified mitigation benefits.

The processes of relicensing existing projects and the licensing of newly developed projects allows the amendment of mitigation requirements which incur costs and often reduce energy capacities. The U.S. Department of Energy (DOE) has initiated a study of environmental mitigation at hydroelectric projects because of concerns relating to this loss of energy capacity and the associated costs. This study is ongoing and study results are periodically reported. Identifying and measuring both the costs and benefits of all types of hydroelectric environmental mitigation is the study's intent. However, only the results to date of investigating fish passage/protection mitigation costs at hydroelectric projects will be discussed in this paper.

The first report (Volume I. Current Practices for Instream Flow Needs, Dissolved Oxygen and Fish Passage) was published in December 1991. Volume I reported cost information that was obtained from 141 projects. A "broad brush" approach provided information on costs and practices. This research indicated a need to closely examine specific practices and cost drivers. Volume II Fish Passage/Protection, which is ongoing, has taken a case study format to more closely examine the costs incurred in association with fish passage/protection mitigation methods at hydroelectric projects. It is anticipated that the Volume II report will be available for distribution at Waterpower '93.

VOLUME I. FISH PASSAGE/PROTECTION COSTS

Cost information was obtained directly from non-federal hydroelectric developers. The developers voluntarily provided this information which was indexed to 1991 dollars. Capital and study costs are considered to be one-time, non-reoccurring costs and are presented as averages per project and dollars per kilowatt of capacity. The operations and maintenance, and reporting costs are considered annual, reoccurring costs and are presented as averages per project and mills per kilowatt hour (kWh) of energy. Lost generation data was also obtained, however, it is difficult to qualify. Some hydroelectric players may not view that part of a waterway that is reserved for minimum and/or bypass flows as a resource that is available for generation, thus they reason that no generation loss occurs. It is difficult to determine whether an entire water source represents potential energy or if only a partial quantity of water is available and there is not a loss of energy.

Because space limitations preclude a thorough discussion of generation losses, this data is not presented here to avoid misrepresentation. It is available in the Volume I report.

Volume I discusses the various costs and practices by capacity categories. The numbers of projects reporting each cost is also provided. Figures 1, 2, 3 and 4 provide an overview of the average costs per project, costs per kilowatt of capacity, and costs in mills per kWh for all projects reporting upstream and downstream fish passage/protection related mitigation costs. Not all of the 141 projects reported each type of mitigation requirement and cost. The number of projects providing each type of cost is provided in the Volume I report.

A few large projects can influence the overall average cost for a particular practice so it is imperative to understand the presentation mode and limitations when viewing the graphs. Because of space limitations, only composite averages are provided. However, to highlight the benefit of viewing costs in capacity ranges the following example is provided. Thirty-seven projects reported downstream fish passage/protection capital costs at an average of almost $1 million per project (see Figure 2). The two projects in the over 100 MW capacity category reported an average capital cost of $12.9 million for downstream fish passage/protection. The twelve projects in the smallest capacity category, under 1 MW, reported an average capital cost of $26 thousand for downstream fish passage/protection. The downstream fish passage/protection capital costs average $17 per kilowatt of capacity (see Figure 1) and they ranged from $14 per kilowatt for the over 100 MW category, to $80 per kilowatt for the smallest capacity category. Significant variations in practices exist, including the use of simplistic angle bar racks, complex tilting traveling screens, to bypass systems that have been mined through the interior concrete of large dams. This variation in practices as well as the range of project proportions drive the ranges of costs.

The upstream fish passage/protection methods also exhibit significant variations in costs associated with the different capacity categories. The average upstream fish passage/protection capital cost per kilowatt of capacity is $31.85 (see Figure 1) and the cost range is from $31.62 for the 100 MW and larger projects, to $107 per kilowatt of capacity for the smallest category (< 1 MW). Figures 1 and 2 highlight the higher costs required for upstream passage/protection compared to downstream passage/protection. This variance is representative of the wide use of capital intensive fish ladders for upstream passage/protection, while spill flows and other practices with diminished capital requirements are often used for downstream passage/protection.

Figures 3 and 4 report the annual operations and maintenance (O&M) and reporting costs for upstream and downstream fish passage/protection average costs. Practices and capacities again influence the cost ranges. For instance, O&M costs for downstream fish passage/protection average 0.52 mills per kWh (see Figure 3)

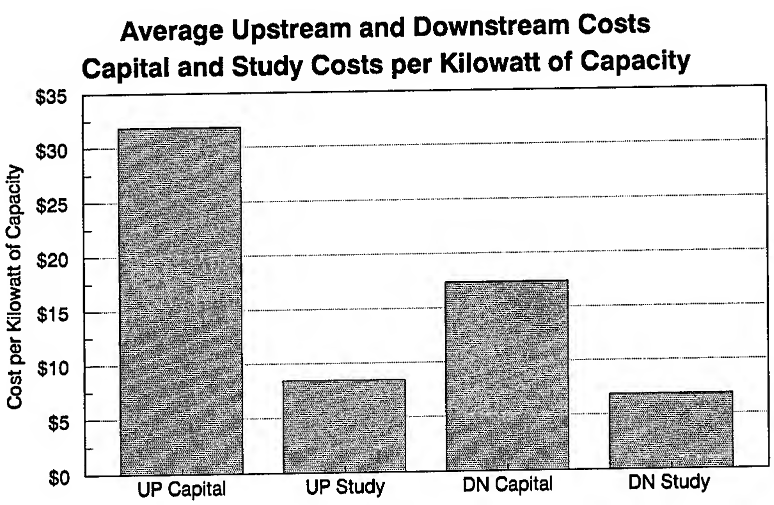

Figure 1. Volume I results, average capital and study costs per kilowatt of capacity, for all types of upstream (UP) and downstream (DN) fish passage/protection methods.

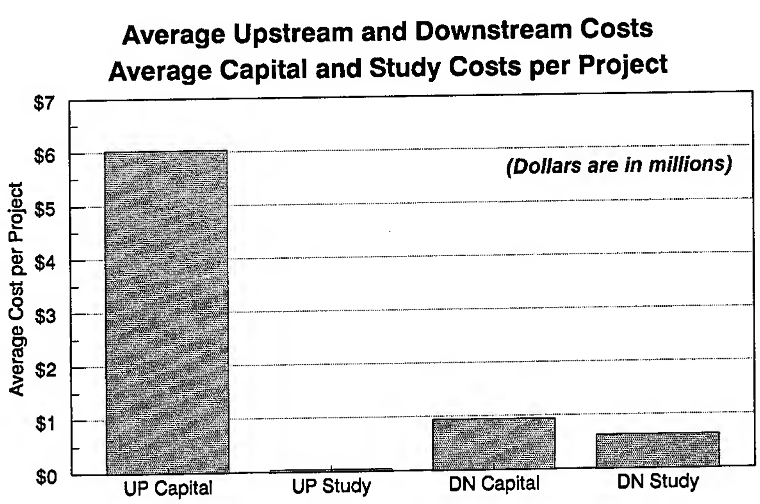

Figure 2. Volume I results, average capital and study costs per project, for all types of upstream (UP) and downstream (DN) fish passage/protection methods. Dollar values are in millions.

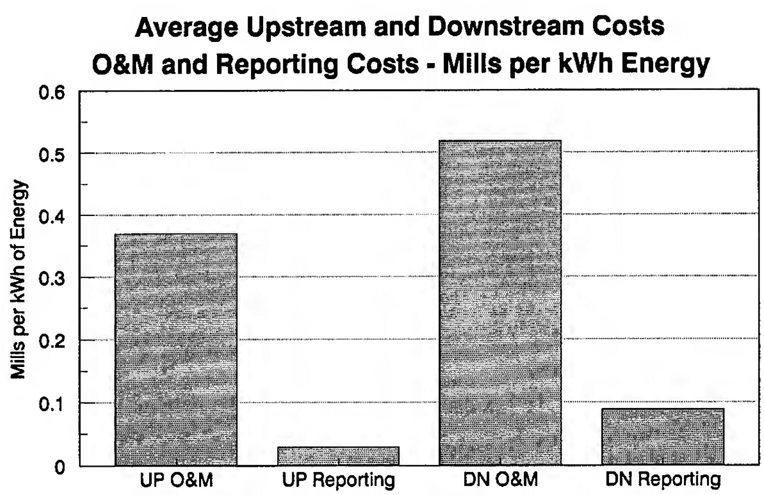

Figure 3. Volume I results, operations and maintenance (O&M) and annual reporting costs, in mills per kilowatt hour (kWh) of energy, for all types of upstream (UP) and downstream (DN) fish passage/protection methods. Costs are assumed as annually occurring.

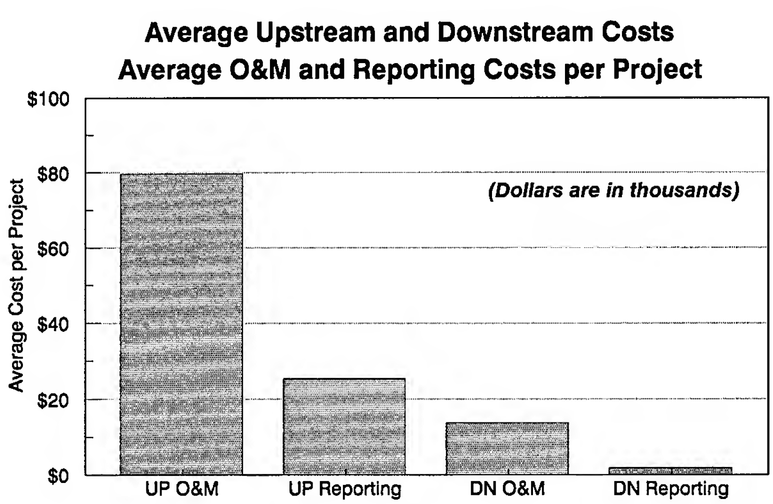

Figure 4. Volume I results, average operations and maintenance (O&M) and annual reporting costs per project for all types of upstream (UP) and downstream (DN) fish passage/protection methods. Costs are assumed as annually occurring.

while ranging from 0.41 mills per kWh to 2.92 mills per kWh for the various capacity categories. The specifics of practices and costs are described in Volume I and a copy can be obtained from any of the authors of this paper.

VOLUME II. FISH PASSAGE/PROTECTION COSTS

Volume I identified the need to examine several mitigation issues in detail. The issue of fish passage/protection is probably the most contentious mitigation subject facing the hydroelectric industry today. It was for this reason that fish passage/protection mitigation issues was selected as the area of focus for the Volume II report. In the effort to more closely examine the costs, practices, and benefits of fish passage/protection, a case study analysis was chosen as the research method for Volume II. This effort is ongoing at the time of this paper's writing, however, some preliminary information is available.

Time and monetary constraints, as is often the case, drove the selection of the number of cases for study. Information was requested from each FERC regional office describing the types and frequencies of mitigation methods used at hydroelectric sites. Preliminary FERC provided data identified approximate methods and frequencies of mitigation. This preliminary data was used to identify the types and frequencies of mitigation practices employed regionally. This information drove the selection of 20 cases nationwide that meet the following selection criteria: mitigation type, capacity size, the availability of information from past studies and reports that identified benefits, the type(s) of resident or migratory fish, the need to obtain a regional sample based on FERC regions and states, and an ownership mixture of entrepreneurs, utilities, municipalities and federal projects.

Several of the original 20 developments identified as viable case studies declined to participate for several different reasons. The remaining 16 developments agreed to participate as anonymous case studies. A geographical sample (see Figure 5) was obtained based on the previously listed criteria. Each site has been visited and data has been obtained describing costs and practices. Several iterations of data collection is often required to most accurately identify all relevant costs and practices. All historical cost data will be indexed to 1993 dollars. The cost analysis for the cases is ongoing at the time of this paper's authorship and it would be presumptuous to report potentially misleading cost data. This information will be available at the Waterpower '93 conference.

The preliminary mitigation frequencies have been elucidated by several iterations of data collection with each FERC regional office and appear to be driven by several factors. The strongest influence of fish passage/protection requirements appear to be site location on a waterway with a migratory or a highly valued resident fishery. The date a project was licensed or relicensed appears to also influence the likelihood of having mitigation requirements. These factors are being further researched. The Chicago FERC region is a good example of the influence

Volume II Fish Passage/Protection Case Study Sites

Figure 5. Participating 16 case study developments, selection based on types and frequencies of regional mitigation methods, fish species, type of ownership, and capacity.

of siting and fish presence to mitigation requirements. The only reported upstream or downstream passage/protection requirements reported in the FERC Chicago region are all fish ladders located on tributaries of Lake Michigan. Lake Michigan area recourse agencies such as the Michigan Department of Natural Resources has placed significant emphasis on improving habitat and passage/protection for the salmon and steelhead trout that migrate via the tributary rivers. Not surprisingly, the Portland FERC region, with its prized salmon and steelhead rivers, has the highest frequency of mitigation requirements (see Figures 6 and 7) of all of the FERC regions.

Upstream fish passage/protection mitigation methods consolidated in the upstream others category (UP Others - Figures 7 and 8) include: passage via navigation locks, tailrace screens, a barrier and passage channels and fish pumps. Some of the downstream fish passage/protection mitigation methods grouped in the other category (DN Others - Figures 7 and 8) include notched wooden boards and barrier nets. The categories of upstream with no mitigation methods (UP NONE) and downstream with no mitigation (DN NONE) are the highest categories of the 1825 responses, regardless of the region.

Additional analysis will examine individual state frequencies, the potential trends based on the association of mitigation requirements to the date of project licensing,

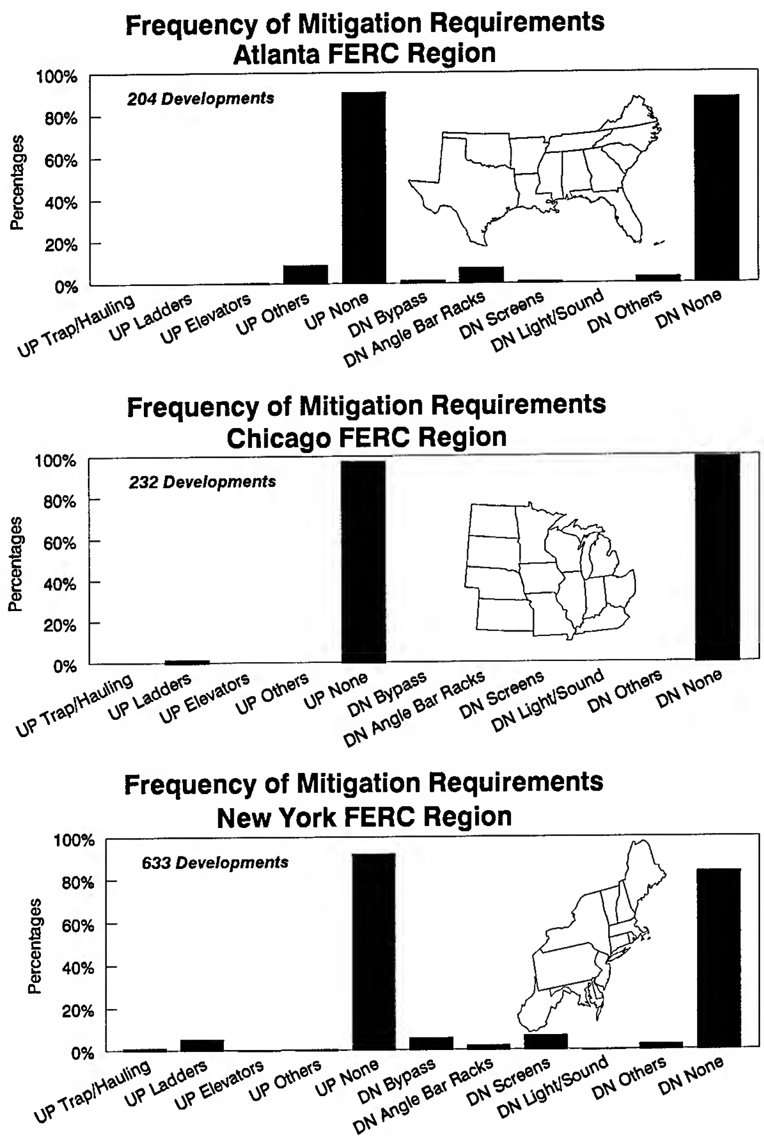

Figure 6. Volume II frequency results, Federal Energy Regulatory Commission (FERC) regional mitigation requirements. Upstream (UP) and downstream (DN) fish passage/protection data obtained from FERC regional offices.

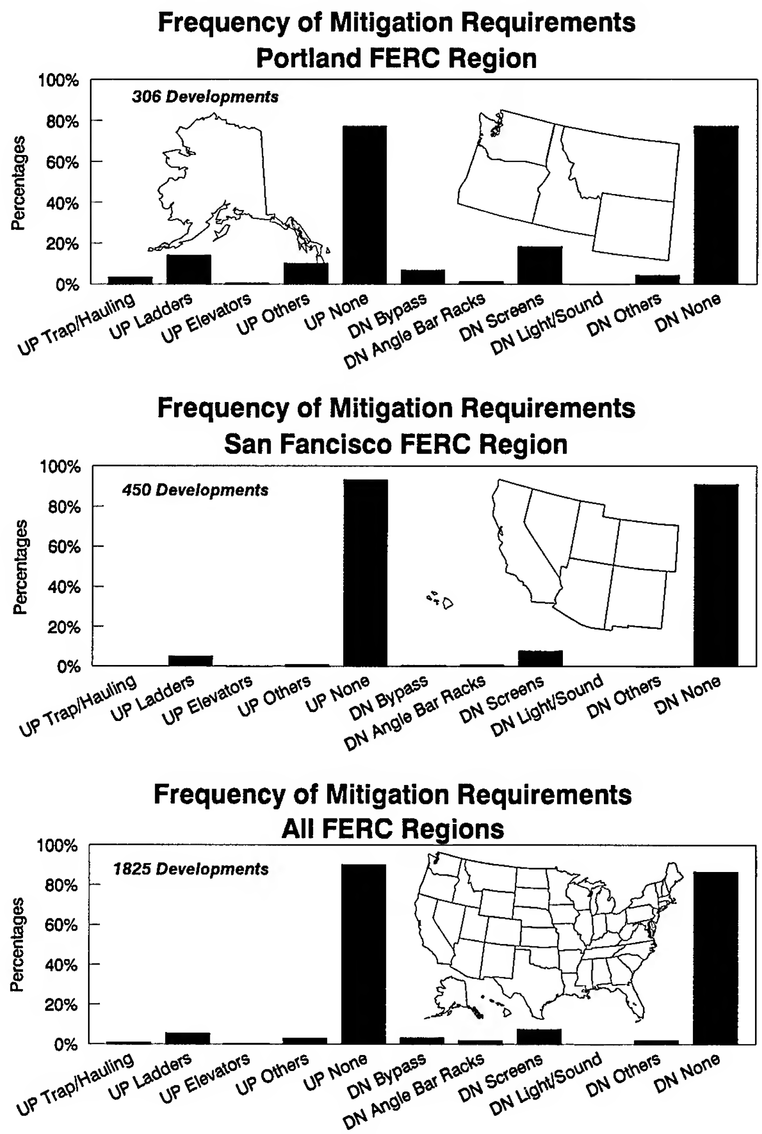

Figure 7. Volume II frequency results, Federal Energy Regulatory Commission (FERC) regional and national mitigation requirements. Upstream (UP) and downstream (DN) fish passage/protection data obtained from FERC regional offices.

and any correlations of the type of ownership to the imposition of mitigation requirements.

DISCUSSION

Significant consultation will continue within each group, and between the hydroelectric industry, FERC, special interest, and state natural resource agency groups as to when and where hydroelectric development is appropriate, and what types of environmental mitigation should be required. Mitigation costs can be imposed with no definitively measurable benefit. Other issues such as degradation of upstream spawning habitat and ocean fishing practices impact the life-cycle of the fish. These other issues have detrimental impacts no matter how much is spent on mitigation at a hydroelectric site. While not depreciating the importance of environmental mitigation, the costs and practices should be valued in the content of how much positive value the mitigation will provide in relation a specie's entire life-cycle.

The costs of mitigation can be substantial, in the form of millions of dollars for ladders, screens or barging operations, yet we rarely measure the costs of other impacts such as lost hydroelectric generation. While some portion of lost generation can be replaced with conservation, this is only a small portion. The replacement energy source can have unseen environmental pollution, be it exploration or disposal costs, air emissions, or transportation accidents. Because hydroelectric's environmental impacts are of a more localized nature, generation losses are not viewed in the content of a systems evaluation of environmental replacement costs.

Figures 6 and 7 indicate that the actual number of projects with mitigation is relatively low. Data provided by the FERC regional offices indicates that nationally, 87% of all active FERC developments have no downstream passage/protection mitigation and 90.5% have no upstream passage/protection mitigation. This may be reflective of the number of sites with no valuable fishery present or that when the developments were licensed, no value was placed on protection/passage action. This study will continue to examine this and other relevant issues to the hydroelectric industry. Each stage of the study will build on lessons learned during earlier volumes and continue to identify the practices, costs and requirements as an aid to successful hydroelectric development in a environmentally responsible manner.

BENEFITS OF FISH PASSAGE AND PROTECTION MEASURES AT HYDROELECTRIC PROJECTS

Glenn F. Čada and Donald W. Jones[1]

Abstract

The U.S. Department of Energy's Hydropower Program is engaged in a multi-year study of the costs and benefits of environmental mitigation measures at nonfederal hydroelectric power plants. An initial report (Volume I) reviewed and surveyed the status of mitigation methods for fish passage, instream flows, and water quality; this paper focuses on the fish passage/protection aspects of the study. Fish ladders were found to be the most common means of passing fish upstream; elevators/lifts were less common, but their use appears to be increasing. A variety of mitigative measures is employed to prevent fish from being drawn into turbine intakes, including spill flows, narrow-mesh intake screens, angled bar racks, and light- or sound-based guidance measures. Performance monitoring and detailed, quantifiable performance criteria were frequently lacking at non-federal hydroelectric projects.

Volume II considers the benefits and costs of fish passage and protection measures, as illustrated by case studies for which performance monitoring has been conducted. The report estimates the effectiveness of particular measures, the consequent impacts on the fish populations that are being maintained or restored, and the resulting use and non-use values of the maintained or restored fish populations.

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[1] Environmental Sciences Division and Energy Division, respectively, Oak Ridge National Laboratory, P.O. Box 2008, Oak Ridge, TN 37831-6036. Work was performed for the Hydropower Program, Office of Renewable Energy Conversion, U.S. Department of Energy, under contract DE-AC05-84OR21400 with Martin Marietta Energy Systems, Inc. This is Environmental Sciences Division Publication No. 4042, ORNL.

Introduction

Hydropower projects can have serious adverse impacts on fish populations by blocking upstream movements or causing turbine-passage mortality of entrained fish. Although numerous mitigative measures are available to enhance upstream and/or downstream fish passage at hydropower projects, their costs can be very high and their effectiveness may be poorly understood. As part of its mission to promote environmentally sound hydroelectric development, the Hydropower Program of the U.S. Department of Energy (DOE) is conducting a multi-year study of environmental mitigation. The first phase of this study was an examination of mitigation practices associated with three issues: fish passage, instream flow requirements, and dissolved oxygen. This paper summarizes the findings related to fish passage (Sale et al. 1991) and subsequent efforts to estimate the benefits of fish passage mitigation.

Approach

Federal Energy Regulatory Commission (FERC) licensing records [the Hydroelectric Power Resources Assessment (HPRA) and the Hydropower Licensing Compliance Tracking System (HLCTS) data bases] were used to identify nonfederal hydroelectric projects that were required to mitigate environmental impacts related to either upstream or downstream fish passage. Because the data contained in these data bases were not sufficient to evaluate costs and benefits of site-specific mitigation practices, a major effort was made to acquire new information directly from the developers of projects for which fish passage mitigative measures were required. Developers were contacted via mailings and were asked to describe the mitigation measures that were required by their FERC licenses, the extent to which the requirements have been implemented, the amount of performance monitoring, and the success of mitigation requirements in protecting aquatic resources. We contacted 707 developers and received 280 responses, most of which indicated that no fish passage requirements had been mandated. Positive returns were representative of the geographic distribution of fish passage requirements (i.e., most returns came from the Northeast, West Coast, and the Rocky Mountain states).

In addition, state and federal resource agencies with responsibilities for recommending environmental mitigation at hydropower projects were also asked for information. Two or more agencies in each state, as well as the regional offices of the U.S. Fish and Wildlife Service (FWS) and the National Marine Fisheries Service (NMFS), were asked to provide information on fish passage issues. Agencies were asked to list projects with fish passage mitigative measures, to describe their mitigation policies and practices, and to identify any studies that could be used to quantify benefits and costs. Agencies from 34 states responded to the fish passage information requests.

Status of Fish Passage/Protection Facilities

Based on data provided by hydropower developers, upstream fish passage measures are estimated to be required at 11% of the nonfederal hydroelectric projects licensed between 1980 and 1990, whereas downstream fish passage was required at 28% of the projects. Generally, fish passage requirements are more common in the western regions of the United States than in the East. The percentage of newly issued licenses that have upstream fish passage requirements did not change significantly over the 10-year period. However, the percentage of new licenses that have downstream fish passage requirements increased from 22% in 1980-83 to 35% in the latter part of the decade (Sale et al. 1991).

Upstream Fish Passage

Most upstream passage measures can be placed into three general categories: trapping and hauling, fishways, and fish lifts. Descriptions of the basic types of upstream fish passage measures are provided in Clay (1961), Hildebrand (1980), and Orsborn (1987).

Information on 34 projects that have upstream fish passage facilities was obtained from hydropower developers. More than 90% of these facilities were either in operation or completed. Fish ladders are the most common mitigative measure, accounting for more than 70% of the upstream passage devices reported. Fish elevators and trapping and hauling are less common.

Performance objectives are essential to assessing the benefits of a fish passage facility. Fifty percent of the respondents indicated that "no obvious barriers to upstream movement" was the only criterion used to judge effectiveness (Figure 1). One facility was required to pass a specified percentage, and one facility was required to pass a specified number of migratory adults. Thirteen percent had some other performance criterion, often relating to the goals of a larger fishery restoration program. Operators of one third of the projects were unaware of any performance objective for the mitigative measure.

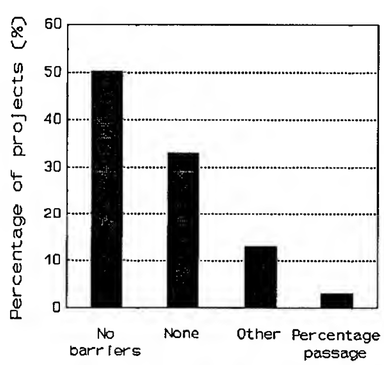

Figure 1. Performance objectives for upstream fish passage

It is important to monitor the operational performance of fish passage facilities in order to make an objective evaluation of site-specific mitigation effectiveness. Performance monitoring at nonfederal hydroelectric projects is relatively rare. Among the 30 operating projects that provided information, 17 (57%) have not monitored the performance of the upstream fish passage measure (Figure 2). Those projects that have monitored the success of upstream passage generally quantify passage rates or, less commonly, fish populations. Forty percent of operating facilities monitor fish passage rates; these are generally fishway counts that are conducted by either the licensee or a fishery resource agency. Monitoring studies that only determine the number of fish that passed through the facility provide an incomplete picture because information about the numbers of fish that did not use the facility (e.g., were unable to find the entrance to the fish ladder) is lacking. Population monitoring studies provide a longer-term view of the success of a mitigative measure because they can estimate whether the fish populations have been maintained or enhanced during the operation of the facility. Twenty three percent of the respondents monitor the fish populations that are protected by the mitigation measure.

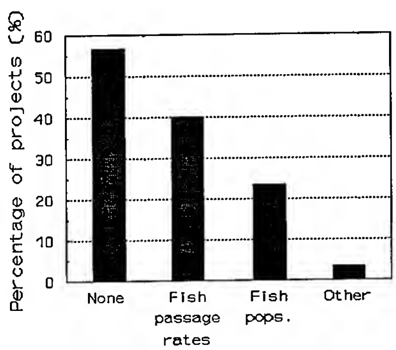

Figure 2. Performance monitoring of upstream fish passage measures

Downstream Fish Passage

Extensive reviews of downstream fish passage mitigation measures are available (Taft 1986; EPRI 1988; Bell 1991). There has been a great variety of measures utilized to reduce turbine entrainment, including spill flows, fixed screens, traveling screens, barrier nets, and sound- or light-based guidance measures. However, no single fish protection system or device is biologically effective, practical to install and operate, and widely acceptable to regulatory agencies.

Information was obtained from 85 hydroelectric projects that have downstream fish passage requirements. The required measures are in operation at 68% of the projects. The single most frequently required downstream fish passage device is the angled bar rack, which is a trash rack that has closely spaced bars (ca 2 cm) set at an angle to the intake flow. Angled bar racks are used by 38% of the projects that have downstream

passage facilities and are especially common in the Northeast. Other types of fixed fish screens are found at 34% of the projects and traveling screens were installed at three of the projects (4%).

Seventy percent of the developers reported that no performance objectives had been specified for the mitigative measure (Figure 3). Four facilities (6%) were required to exclude a specified percentage of fish from entrainment, and three facilities (4%) were required to limit mortality of downstream migratory fish to a specified level. Twenty percent had some other performance objective, usually a qualitative goal such as "effective operation."

Performance monitoring for operating downstream fish passage facilities at the nonfederal projects examined in this study was rare (Figure 4). No performance monitoring was reported at 79% of the 66 projects that have operating downstream fish passage measures. Among the 14 projects that have conducted operational monitoring, 11 monitored passage rates, 10 estimated mortality rates, and 1 monitored fish populations.

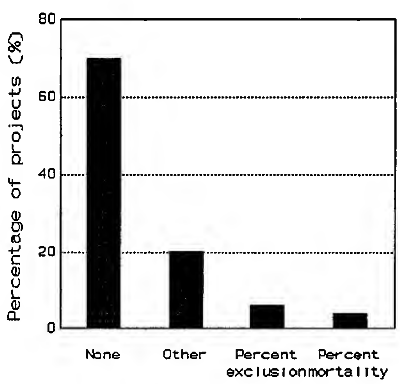

Figure 3. Performance objectives for downstream fish passage measures

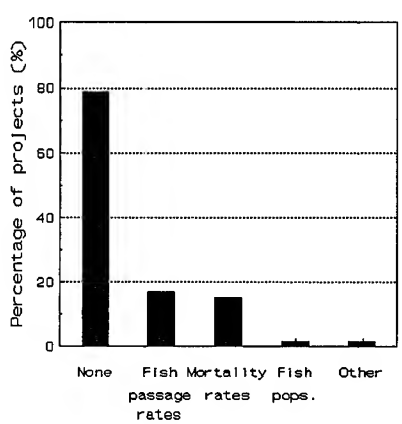

Figure 4. Performance monitoring of downstream fish passage measures

Estimating the Benefits of Fish Passage Mitigation

Whereas Volume I surveyed hydropower mitigation practices and policies for three common environmental issues, i.e., instream flow releases, water quality, and fish passage, Volume II focused solely on the latter issue. In addition to developing more precise estimates of the costs of particular fish passage and protection measures, the report attempted to quantify

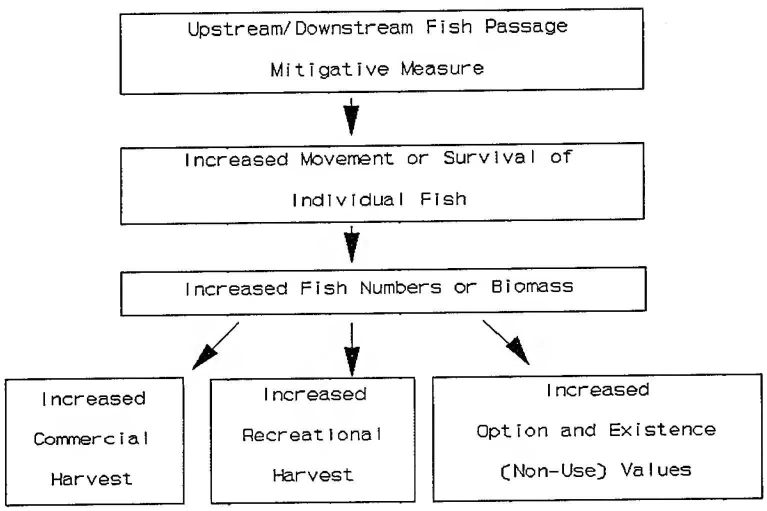

Figure 5. Steps in the quantification of benefits from fish passage mitigation

benefits in economic terms. The benefits of fish passage/protection measures can be estimated through a series of steps illustrated in Figure 5. For example, successful operation of a fish ladder or lift would increase the number of individual fish that are able to move upstream. This, in turn, would increase the number of fish that populate areas upstream from the hydropower dam, either because the fish continue to reside in the newly available habitat or because they reproduce in formerly unutilized spawning habitat. These increases in fish population numbers or standing crop (biomass) may have commercial, recreational, or non-use values that can be expressed in economic terms. Similarly, intake screens may increase the survival of resident or downstream-migrating fish by reducing turbine passage mortality. If the increased survival results in increased fish population numbers or biomass, economic benefits may be realized.

In order to conduct cost-benefit analyses of fish passage/protection measures, values of the fish must be estimated. The values of fish that are harvested commercially (market values) are relatively easy to derive, but estimating other use values (e.g., recreational fishing) and non-use values are much more difficult. For example, the value of a recreational fishery is a

complex function of not only the number of fish available, but also the number of anglers, the amount of money anglers are willing to spend to fish, the number of alternative fishing sites, and other qualities of the river (e.g., scenic beauty, remoteness) not directly related to the supply of fish. Enhancement of the values of a recreational fishery brought about by a mitigative measure may not have a one-to-one relationship with the additional numbers of fish produced.

Even more complicated is the concept of non-use values, which can be divided into option values and existence values. Option value involves the possible consumption of a resource in the future; it is the amount of money that an individual will pay today to assure the ability to fish in the future, over and above the later, use value expected to be derived from recreational fishing. Existence value is the value that an individual attaches to the simple existence of a natural resource even though he has no plans to consume or otherwise use it, including even viewing it. Existence values might be attributed to endangered species (that have no present use or option value) or to biodiversity in an area remote from the individual. Also, individuals who have no intention of engaging in fishing either now or in the future may still attach existence value to the restoration of a salmon run. The Volume II report further discusses these concepts of valuing benefits and describes empirical approaches to estimating them.

As reported in Volume I, monitoring of fish passage/protection measures that would permit an estimate of benefits has been relatively rare. Most nonfederal hydropower projects have not monitored changes in distribution or survival of fish (the first step in assessing benefits in Figure 5), let alone the resulting changes in fish numbers/biomass or the changes in use and non-use values. In view of the relatively low degree of performance monitoring, analyses in Volume II relied on evaluation of a small number of projects (case studies) that have collected data that could be used to assess benefits. These case studies were selected to encompass the widest range of mitigative measures, fish species, and geographic regions possible (Table 1). Few studies have been conducted in the Midwest or Southeast, or on river systems which support only resident fish. As a result, most case study sites were located in the Northeast or the Pacific Coast, and nearly all monitoring data concerned salmon or other anadromous fish. Fish ladders were the subject of most upstream fish passage case studies, which reflects the preponderance of ladders as a mitigative measure at nonfederal hydropower projects (Sale et al. 1991). Although a wide range of downstream protection measures is employed by the hydropower industry, performance monitoring has been carried out most extensively at sites with some type of fixed screens, e.g., angled bar racks, inclined screens, or wedge-wire screens.

Evaluation of the benefits of the mitigation at each case study site was based on a review of published performance monitoring data. Case study descriptions included characteristics of the hydropower project and the mitigative measure, the environmental setting, and the fish resource

Table 1. Mitigative measures used at case study projects to enhance fish passage or to reduce turbine passage mortality.

Mitigative Measure	Fish Species	State
Fish ladder	Steelhead trout Chinook salmon	MI
Fixed intake screens	Blueback herring	NY
Angled bar rack	Atlantic salmon	NY
Fish ladder	Atlantic salmon American shad Alewife	ME
Fish ladder Fixed intake screens	Atlantic salmon	ME
Fish ladder Fish lift	Atlantic salmon American shad	MA
Fish lift	American shad	MD
Fish ladder Wedge-wire screens	Chinook salmon Steelhead trout	CA
Fish ladder Fixed intake screens	Rainbow trout	CA
Fish ladder	Chinook salmon Steelhead trout	CA
Fixed intake screens	Chinook salmon	OR
Fixed intake screens	Chinook salmon Steelhead trout	OR
Wedge-wire screens	Chinook salmon	OR
Fish ladder Spill flows	Chinook salmon Sockeye salmon Steelhead trout	WA
Fixed intake screens	rainbow trout cutthroat trout brook trout	WA
Fish ladder Traveling screens	Chinook salmon Steelhead trout	WA

management goals and objectives for which the mitigation was designed. The performance of the measure was compared to stated management objectives or license conditions. In most case studies, the benefits of the measure could only be expressed in terms of the numbers of individual fish that were transported around the dam or protected from entrainment. As might be expected from the types of monitoring that are most commonly conducted (Figures 2 and 4), population-level responses of the target fish species were rarely known.

All parties to hydropower development must have an accurate understanding of both the cost and benefits of fish passage mitigative measures. Construction and operation of often costly fish passage measures may be required at sites where the need is uncertain (e.g, at sites without clearly migratory fish species) or where the subsequent biological benefits remain unknown. Wherever possible the value of fish potentially transported around an impassable barrier should be quantified and compared with construction and operation costs of mitigative measures, to ensure that costs do not greatly outweigh benefits. Obviously such comparisons must be made with caution because the value of species that are being protected from extinction or are undergoing restoration may not be easily expressed in economic terms.

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A PROGRAM TO IMPROVE FISH SURVIVAL THROUGH TURBINES

John W. Ferguson [1]

Abstract

Fish protection at hydroelectric facilities often requires the construction of fish passage facilities. Benefits from juvenile fish bypass systems have not been rigorously evaluated. In one recent study, the juvenile bypass system produced lower survival rates than turbines. Turbine designs that provide safer fish passage conditions could increase fish populations, reduce the need for mitigation, and reduce regulatory pressures on the hydro-industry. A program to define the mechanisms of mortality acting on fish and develop biologically based turbine design criteria is presented.

Introduction

The examination of fish mortality associated with turbine passage was initiated in Sweden in 1927, where injured salmon and eel were observed having passed through turbines on the river Gota alv. The early investigations that followed documented the extent and type of injuries. Similar investigations were conducted in Europe through the 1960's to understand the magnitude and ascertain the causes of mortality (Monten, 1985). In the western United States studies were conducted on juvenile salmon at Columbia River dams starting in the late 1930's (Holmes, 1952). Schoeneman et al. (1961) estimated turbine mortality at McNary Dam, and Oligher and Donaldson (1966) estimated turbine mortality at Big Cliff Dam on the North Santiam River, Oregon. Long et al. (1968) estimated losses through Ice Harbor Dam on the Snake River. The above studies are typical of turbine mortality investigations designed to

[1]Fish Biologist, Portland District, Corps of Engineers, P.O. Box 2946, Portland, Oregon 97208-2946

provide estimates of turbine mortality, and many compared survival rates through various dam passage routes. A few of the studies examined the relationship between fish survival and certain design or operational variables. For example, Oligher and Donaldson (1966) found that fish survival generally increased with unit efficiency. None examined the causal agents of the mortality.

Recently, research efforts on the Columbia River have focused on removing fish from turbine intakes using guidance screens. This led to the development of the submersible traveling screen to guide juvenile fish into bypass systems, and shifted research away from understanding turbine mortality mechanisms, to bypass system development.

However, the benefits associated with bypass systems have not been rigorously tested. According to Hydro Review (June 1992, Tech Briefs), a U.S. Department of Energy study conducted by the Idaho National Energy Laboratory concluded that "despite the high cost of environmental mitigation at hydro-electric projects, there has been little data collected on its effectiveness." The report concludes that mitigation benefits are essentially unknown.

At Bonneville Dam on the Columbia River Ledgerwood et al. (1990) used estuary recoveries of juveniles and adult returns to estimate mortality rates for various passage routes. They concluded that fish released into the bypass system had significantly lower survival than fish released into the turbine, spillway, turbine discharge, and downstream from the project. Subsequent investigations suggest the bypass system causes fish to be stressed and fatigued, possibly increasing their susceptibility to tailrace predation. The study questions the assumption that bypass passage is better than turbine passage. After reviewing additional information on bypass systems, Ferguson (1992) concludes that comprehensive analyses of bypass systems compared to other passage routes are warranted.

In summary, the current information is limited primarily to the estimation of fish mortality, not the causal mechanisms of mortality. Precise information on the actual mechanisms of mortality and what can be done to design new or rehabilitate existing turbines to provide a safer passage environment for fish is unavailable. Current efforts are focused on mitigation, the benefits of which have not been rigorously tested.

Recommended Program

The goal of the program outlined below is to produce new turbine designs that improve fish passage conditions and reduce regulatory pressures on the hydropower industry. Biological design criteria, developed from a thorough and fundamental understanding of the mechanisms of mortality, could be used to develop new turbine designs.

A program that defines the mortality processes associated with fish passage through a turbine and develops biological design criteria has the following objectives:

1. Gain a thorough understanding of the mechanisms of fish mortality.

2. Define the biological requirements or sensitivities of key fish species to these mechanisms of mortality.

3. Develop new turbine design criteria to reduce fish mortality.

4. Construct prototype turbine designs and test for fish passage, hydromechanical, and power production.

5. Identify construction and power costs associated with new turbine designs.

The mechanisms currently thought to be the causal agents of fish mortality within turbines are:

1. Strike: the probability and effect of fish impacting stay vanes, wicket gates, and runner blades.

2. Pressure: the effect on fish of passing through the pressure environment of a reaction turbine, especially associated with the pressure drop on the suction side of the runner.

3. Cavitation: the effect on fish of passing in a region of cavitation, which is the implosion of water vapor pockets produced by the pressure environment in some areas of the turbine.

4. Shear: the effect on fish of encountering rapidly changing water directions and associated hydraulic forces.

5. Stress: the effect on fish of passing through the turbine environment, inducing a debilitating level of stress, and weakening the animal's resistance to disease and predation.

6. Grinding: the loss of fish through the narrow gap between blade tips and the discharge ring.

A program to understand how each mechanism contributes to overall mortality is outlined below.

<u>Strike</u>

Strike is considered an important component of overall passage mortality. The effect and probability of strike are governed by many variables including fish length, unit rpm and discharge, number of runners, and the angle at which fish approach the runners. The angle where non-lethal strikes become lethal and design changes that may reduce strike need to be defined.

To investigate strike, video cameras would be installed in hydraulic models of turbines to observe the behavior of neutrally buoyant particles or small fish. In 1993, the U.S. Army Corps of Engineers will install a Kaplan turbine model in a hydraulic model at the Waterways Experiment Station (WES) in Vicksburg, Mississippi. Medical industry video cameras could help define the relationship between wicket gate setting and runner strike, approach angle and strike, and the relationship between vertical fish distribution in the intake and horizontal distribution across the runner.

Under prototype conditions low-light video cameras could be flush-mounted in the speed ring to observe fish behavior. The horizontal distribution across the runner could evaluate the hypothesis that in large turbine intakes, the intake guidance screens may be deflecting unguided fish deeper into the intake, where passage past the distal area of the runner may increase mortality.

Combining model and field data would suggest possible design changes needed to minimize strike. Directional fish guiding vanes, modifications to the runner shape, and other changes could possibly reduce the probability and effect of strike. Turbine manufacturer's could model fish design effects on the electrical-mechanical aspects, efficiency, and power production. A prototype design could be constructed and field tested.

Pressure

The primary sources of pressure loss are thought to be swim bladder ruptures and gas bubble disease. Swim bladders can be damaged or ruptured when air in the bladder expands rapidly as fish pass through areas of below atmospheric pressure. Nitrogen from air in the swim bladder or nitrogen in tissue or fluids can leave the tissue or solution and cause death through an embolism in a vital organ such as the heart.

Bell at al. (1991) estimates pressure losses relative to fish depth. The deeper the depth fish are accustomed to, the greater the loss when they encounter the pressure drop in the turbine environment. Physostomous fish such as salmon have ducts connecting the air bladder and esophagus and are less susceptible to pressure changes than physoclistic fish which lack the duct. Bell believes that runners designed to maintain pressures above 0.5 atmosphere would provide a relatively safe fish passage environment (Personal Communication, Milo Bell, Mukilteo, Washington).

A laboratory research program should retest the pressure loss concepts with modern, rapid, pressure-drop technologies. Various sizes, species, and conditions of fish (descaled, diseased, healthy) could be examined, along with the effect the depth at which fish are accustomed has on survival. In particular, the effect high total dissolved gas levels within the fish has on survival should be investigated. Once the pressure criteria are defined turbine manufacturers could incorporate the biological design criteria into their models and design runners that meet these criteria.

Cavitation

Cavitation is an extreme form of the pressure effect within turbines. Turbines are designed to reduce cavitation to levels considered acceptable from engineering and maintenance perspectives. However, the amount of cavitation which remains in the passage environment may affect fish survival. The effects of cavitation are largely unknown, although the pressure wave associated with the implosion of the vapor pocket is similar to that of the shock wave produced by underwater blasting, which can be harmful to fish.

Any effect anti-cavitation fins may have on fish should be investigated. Anti-cavitation fins are installed at many sites to move cavitation away from the blades to reduce blade maintenance. Since they simply move but do not decrease cavitation, from a fish passage standpoint, they seem misnamed. Regardless, there is an untested concern that they move cavitation into fish passage routes and may increase fish mortality. Laboratory tests could define

cavitation design criteria.

Shear

Shear is probably the least understood and least identifiable agent of mortality (Eicher at al., 1987). While difficult to research, the influence shear has on passage survival should not be overlooked. Current activities designed to mitigate for turbine passage may possibly increase the level of shear and the incidence of shear effect. Experimental Columbia River juvenile fish guidance screens in Kaplan unit intakes redistribute and accelerate flow toward the bottom of the intake. This may increase the level, incidence, and effect shear has on unguided fish passing through the unit.

The laboratory environment could be used to develop tolerance criteria. To link the tolerance criteria to conditions found in operating units, laser doppler measurements of velocity in hydraulic turbine models and computational fluid dynamics modeling of turbine shear should be attempted. Once shear criteria have been determined for the species of interest and the level of shear for existing units documented, turbine designers could develop alternative designs that reduce shear. Effects on power production and unit efficiency could be assessed for a selected design.

Stress

Stress associated with passage through juvenile fish bypass systems has been evaluated only recently. Researchers found the performance of mitigation facilities often needs to include an assessment of physiological factors associated with the generalized stress response. The analysis of blood parameters such as cortisol and lactate, metabolic fluxes, and osmotic balance can be useful tools in designing effective passage facilities (Schreck, 1991).

Similarly, the analyses of stress associated with passage through turbines can be a useful tool in designing effective turbine passage environments. The implications of stress effects are not clearly understood, although the available information indicates stress can influence endocrine, osmoregulatory, metabolic, and immune system function. For example, recent studies indicate that juvenile Pacific salmon have an increased vulnerability to predation when stressed.

Initial laboratory investigations would define the level of stress a species could tolerate. Selected prototype turbine design could be field tested and stress parameters associated with passage through the prototype measured and compared to the criteria for conformance.

Grinding

The potential for mortality associated with passing through the gap between the distal end of the runner and the discharge ring is high. Fish could be drawn to this area by the high velocity of the leakage past the tip, and once there are likely damaged when passing through the small opening. Eicher et al. (1987) speculate that the level of mortality in large Kaplan turbines (between 4% and 15%) is roughly the same as that percentage thought to pass through the peripheral runner gap.

Video camera imaging under prototype conditions could identify the level and outcome of gap passage. If found to be a significant problem, designs could be developed utilizing flow vanes or other measures to redirect fish away from the gap area.

Benefits of a Turbine Mortality Research Program

A program that quantifies the mechanisms of turbine mortality, provides biologically based design criteria, provides an understanding of associated power impacts and engineering considerations, and leads to new turbine designs could provide numerous benefits, including:

1. Increased populations of certain species and stocks of fish.

2. A potential reduction in regulatory pressures on the hydropower industry including the easing of restrictions on existing projects, the possible opening up of new areas for development, and a possible reduction in the need for mitigation measures.

3. The future existence of power producing facilities if current fish mitigation activities do not provide adequate levels of protection, and decisions are made which favor fish protection over power production.

The most obvious benefit from the suggested program is to fish populations, including endangered species or stocks. Each region has its own species of concern. In the Pacific Northwest, one alternative being considered to protect salmon is the return of reservoirs to natural river bed flow. This underscores the seriousness of the actions being considered under the current regulatory environment (ESA) when valuable fisheries stocks become threatened or endangered, and turbine mortality rates are considered to be unacceptably high. It also indicates the magnitude of the potential benefit associated with providing safer fish passage conditions through turbines.

Turbine rehabilitation programs could also incorporate new runner designs that provide safer fish passage conditions. The benefits would be increased passage survival, and in certain cases an increase in power production through a reduction in the amount of water spilled for mitigation.

Understanding turbine biological design criteria may benefit the future existence of power producing facilities in a regulatory environment where fish species or stocks are considered threatened or endangered under ESA, and existing mitigation programs do not provide adequate protection. Mitigation measures may not provide the level of protection necessary to meet regulatory requirements. Bypass systems will hopefully prove effective in restoring fish runs, but after complete evaluation they may be found ineffective due to additional mortality factors such as stress or predation at bypass release points. The choice becomes power production with changes made to the turbines to protect fish, removal of the dam, or reconfiguration of the dam to pass flow via a natural river channel without power production during the fish migration period.

The Corps is considering operating key reservoirs at elevations significantly below the designed operating range. These actions attempt to reduce juvenile salmon reservoir mortality by increasing water particle travel speed, which presumably reduces predation by increasing the rate at which salmon travel through the reservoir. Little is known of the effect pool lowering has on the turbine passage environment. Based on Oligher and Donaldson (1966) fish mortality may increase as a result of the estimated 5% reduction in unit efficiency. Understanding the mechanisms of mortality would improve overall assessment of mitigation measures such as pool lowering.

How fish are distributed along a runner is needed to assess the potential impact of intake mitigation screens. Intake guidance screens alter intake flows and deflect unguided fish deeper into the intakes, possibly distributing fish toward blade tips, which is thought to be an area of higher mortality.

Conclusions

The information discussed in this paper has a basis toward large Kaplan turbines and salmon passage issues. Granted, most of the new hydropower facilities being developed are small to medium in size. However, much of the current knowledge on passing fish around hydro-projects has been developed at large dams, and the technology and information applied to smaller facilities. The same principle will be true for turbine redesigns for fish passage. A program developed along the lines suggested will have national utility. The principles learned and biological criteria developed will be useful for all types

and sizes of turbines where fish passage is a consideration.

The goal is to develop a research program that brings the latest technologies together with engineering and fish passage expertise. Engineering and fish passage experts need to work together to eliminate our knowledge gaps regarding fish passage through turbines, and prepare the industry for the inevitable challenges of the future.

Potential benefits from this type of program include increased fish populations, economic development associated with increased harvest of certain species, reduced regulatory pressures on relicensing projects, and in some cases increased power production from a reduction in spill used for mitigation. The costs include a possible reduction in power production caused by the new runner designs. The program may also reduce the potential for returning some projects to natural river flows, an alternative under consideration in the Pacific Northwest.

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Turbine Passage Survival at Low-Head Hydro Projects

John A. Matousek,[1] Alan W. Wells,[1] Kevin G. Whalen,[2]
Jack H. Hecht,[2] and Susan G. Metzger[3]

Abstract

Environmental monitoring programs related to the relicensing of hydroelectric projects have been focused on determining impacts associated with turbine entrainment. A major component of the operational impact evaluation is the assessment of turbine-related mortality. As part of a comprehensive entrainment abundance monitoring program conducted for Consumers Power Company at 11 Michigan hydroelectric projects, a seasonal program to determine turbine passage survival for major resident fish populations was conducted at four projects, with turbine configurations and hydraulic conditions representative of all 11 projects.

Turbine survival testing was conducted during the spring and fall at two projects on the Au Sable River and two on the Muskegon. With the exception of walleye used at the two Muskegon River sites, all fish used in the tests were purchased from a commercial supplier. Fish were fin clipped and held for a minimum of 24 h prior to testing. Fish species were separated into three test groups: one was introduced directly into the turbine, one into the draft-tube collection net. The third group was held in the floating holding nets to determine marking and holding stresses. Initial and extended (48-h) survival observations were recorded for all test fish.

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[1]Program Manager, Lawler, Matusky & Skelly Engineers, One Blue Hill Plaza, P.O. Box 1509, Pearl River, NY 10965

[2]Project Scientist, Lawler, Matusky & Skelly Engineers, One Blue Hill Plaza, P.O. Box 1509, Pearl River, NY 10965

[3]Director, Environmental Sciences Section, Lawler, Matusky & Skelly Engineers, One Blue Hill Plaza, P.O. Box 1509, Pearl River, NY 10965

Testing results show that post-turbine entrainment survival did not differ markedly between seasons, nor did it differ at projects having different physical and hydraulic characteristics. Overall turbine entrainment survival for all four projects was 83.5%, ranging from 79.1 to 86.0%. By species over all projects, survival ranged from 57% (spottail shiner) to 90% (bluegill). Turbine passage was found to induce low to moderate levels of mortality on common fish species resident in the Au Sable, Manistee, and Muskegon rivers.

Introduction

Background

The determination of turbine passage fish survival is important to hydroelectric project relicensing since it establishes a general baseline for the consideration of downstream fish passage requirements. As noted by Ruggles et al. (1990), if turbine passage mortality is low (5 to 15%), then turbine entrainment may have less impact than that associated with intake exclusion and downstream passage techniques. Over the past several years a substantial amount of information has been developed on turbine entrainment and survival, but very little on mortality associated with mitigative strategies and systems (Sale et al. 1991; EPRI 1992).

As part of the relicensing activities at 11 Michigan low-head hydroelectric projects owned and operated by Consumers Power Company, mark/recapture studies were conducted to evaluate turbine entrainment survival on selected fish species. The projects are located on three lower peninsula Michigan rivers: six on the Au Sable, two on the Manistee, and three on the Muskegon, all in the Great Lakes drainage basin.

Program Objective

The overall program objective was to obtain information on the survival of resident fish species following passage through Francis turbines under normal power generating conditions. Specific program objectives were to:

- Categorize the turbine characteristics of the 11 projects and select representative facilities from each category for the evaluation of entrainment survival.
- Evaluate seasonal survival at the representative projects relative to abiotic parameters, including water temperature, and project-specific variables, such as head, turbine size, and orientation.

- Evaluate turbine entrainment survival of representative fish species resident in the respective impoundments relative to biotic variables, including species and length.

Hydroelectric Project Description and Study Site Selection

To determine the representative projects for turbine entrainment testing, each of the 11 hydroelectric projects was placed in one of four groups based on turbine characteristics and operation, specifically, turbine orientation (horizontal or vertical Francis unit), runner diameter, number of blades per runner, revolutions per minute (rpm), head, and peripheral runner velocity (Table 1). From the four groups, the projects selected for testing were Alcona and Five Channels, both located on the Au Sable River, and Hardy and Rogers, both located on the Muskegon River.

Turbine Entrainment Survival Program

Description of Sampling Program

Turbine entrainment survival studies were conducted at the four representative hydroelectric projects during the spring and fall of 1990. The study was designed through consultation with the Michigan Department of Natural Resources, U.S. Forest Service, and U.S. Fish & Wildlife Service.

During each seasonal test period, turbine entrainment survival was estimated by comparing the survival of three groups of fish for each species and project. The three test groups per species included a control group maintained in floating holding nets for the duration of the study, a second control group introduced directly into the mouth of the draft-tube collection net, and the turbine-introduced group. All fish in each group were held in large floating nets constructed of knotless nylon for at least 24 h prior to testing and 48 h following recapture. Mortality in the net-held group may be attributed to handling, marking, and holding; mortality in the collection net control group may be attributed to marking, handling, collection (net effect), and holding. The fish released at the turbine were exposed to all stresses experienced by both groups of control fish as well as turbine-related stresses. Different fin clips or fin-clip combinations were used to distinguish test groups.

With the exception of Muskegon walleye used at Hardy and Rogers, the fish species used in the study were obtained from commercial suppliers. The number tested and seasonal length range are presented in Table 2.

TABLE 2. - Fish Species Tested and Length Range by Season

	Spring Test Period		Fall Test Period	
	Number Tested	Length Range (mm)	Number Tested	Length Range (mm)
Black crappie			13	152-218
Bluegill	571	46-206	1087	58-244
Golden/common shiner			976	62-207
Grass pickerel			68	177-293
Largemouth bass			257	74-162
Northern pike			357	248-456
Rainbow trout	442	57-401	191	81-410
Spottail shiner	252	58-174		
Walleye			734	78-638
White sucker			1013	81-442
Yellow perch	259	27-263	707	61-310

Test fish were received and maintained in age class groups corresponding to juvenile and adult for each seasonal test. After testing, each fish was measured in order to evaluate the data by length-frequency distribution rather than age class.

The entrainment survival monitoring program was generally conducted according to the following steps: Fish in the transportation tanks were acclimated to pond temperatures with pond water, fin clipped, and transported to floating holding nets. Following a minimum 24-h acclimation period, fish to be tested were removed from the holding nets, counted, and placed in plastic containers; up to 30 small fish and from five to 30 large fish were placed in each transport container. All fish releases were made through a 15.2-cm PVC pipe. Holding net control fish were returned immediately to the floating holding net; collection net control fish were discharged near the center of the net mouth; turbine test fish were released into the turbine bay as close to the wicket gates as possible.

Turbine and net-introduced fish were collected in a draft-tube net with a floating box attached to the cod end. At each facility the draft-tube net was attached to a metal frame that was lowered into existing draft-tube stoplog slots; the net was designed to intercept the entire turbine discharge flow from the selected unit(s). The draft-tube nets were constructed of 12.7-

and 9.5-mm untreated knotless nylon mesh with a 6.4-mm untreated knotless nylon mesh liner in the final one-third of the net. Each net mouth's opening dimensions and length were plant specific.

Fish were removed from the floating collection box and sorted by species and fin clip. Their initial condition was recorded (live, stunned, or dead), and dead fish were removed, measured to the nearest millimeter, and checked for internal and external damage. Live and stunned fish were transported to a floating holding net. Extended survival condition was recorded for all fish at 48 h, with intermittent checks conducted at 12 and 24 h from the time of collection.

Dead fish were removed at each check, measured, and checked for damage or cause of death. The condition of all fish remaining in the holding net was recorded at the end of the 48-h period. All fish were measured to the nearest millimeter; dead and stunned fish were checked for external and internal damage. Live fish were released into the tailrace at the end of the test.

Analytical Procedures

Fish were grouped by the life stage identified by suppliers. Turbine entrainment survival was calculated by life stage, using all of the data available from each test group. Some fish were temperature stressed or developed fungal infection during the test period. Survival rates calculated for these groups are believed to be lower than those for unstressed fish.

Estimates of initial survival (fish classified as alive immediately following collection) and extended survival (fish classified as alive at the end of 48 h) were calculated for each species and life stage (juvenile or adult) by test group at each project. At the end of the 48-h extended survival period stressed fish were considered dead in calculating survival. Since some of these fish may have survived, survival rates may be conservatively low.

Initial and extended survival was calculated as follows:

$$\textit{Initial Survival } (S_I) = \frac{\sum \textit{Fish Alive at Recovery}}{\sum \textit{Fish Recovered}} \tag{1}$$

$$\textit{Extended Survival } (S_{48}) = \frac{\sum \textit{Fish Alive at 48 h}}{\sum \textit{Fish Observed for Extended Survival}} \tag{2}$$

Control (S_C), net (S_N), and turbine (S_T) test group survival rates are calculated from initial and extended survival for each test group using the formula:

$$Survival = S_I \times S_{48} \tag{3}$$

Turbine entrainment survival (S_E) was calculated by correcting turbine (S_T) test group survival for control and net survival (S_C, S_N) using the following formula:

$$Turbine\ Entrainment\ Survival\ (S_E) = \frac{S_T}{\left(\frac{S_N}{S_C}\right) S_C} \tag{4}$$

The second phase of the turbine entrainment survival analysis was an assessment of covariate effects. Previous studies (EPRI 1987) have indicated that water temperature, turbine operational characteristics, fish size (age or weight), debris load, head, and species composition influence survival. In general, high water temperature, small body size, heavy debris loads, cavitation, and high peripheral runner velocities tend to decrease survival; low water temperature, mature fish, low or no cavitation, and low peripheral runner velocity tend to increase survival.

To investigate factors that may influence turbine entrainment survival, Formal Inference-based Recursive Modeling (FIRM) (Kass 1980; Hawkins and Kass 1982; Hawkins 1990) was used. This technique is useful for evaluating the structure within large data sets of mixed data types. Given a data set of a single dependent variable (in this study, survival) and two or more independent variables measured on nominal (categorical), ordinal (ranked), or interval (continuous) scales, the procedure successively examines the dependent variable with respect to the various factors while preserving the nature of the scale. Potential influences included in the analysis of survival were hydroelectric project, test group, fish species, total length in 50-mm increments, and season. The dependent variable was a nominal measure, either "live" or "dead."

Program Results and Discussion

Over the two seasonal sampling periods at the four hydroelectric projects, a total of 3145 live fish and 746 dead fish (no dead fish release at Hardy) were released at the turbine; 2656 of the live fish (84.5%) and 691 of the dead fish (92.6%) were recovered in the draft-tube collection net(s). Collection efficiency values for live fish by project were: Hardy, 81.0%; Five

Channels, 82.8%; Rogers, 91.0%; and Alcona, 92.6%. Collection efficiency values for dead fish releases were all greater than 90%, ranging from 90.6% at Rogers to 97.1% at Alcona. At Five Channels the value was similar to Rogers at 91.0%.

Overall turbine entrainment survival, 83.5%, was similar at the four projects, ranging from 79.1% at Hardy to 86.0% at Five Channels (Table 3). Turbine entrainment survival was around 90% for juvenile and adult bluegill at all four projects. Bluegill were tested during both seasonal periods at Alcona, Five Channels, and Rogers, with similar survival values obtained for each season. Turbine survival (fall survey only) was slightly lower at Alcona compared to the other three projects. Juvenile largemouth bass were evaluated at Hardy and Rogers, with similar average survival values (76.2 and 77.4%).

Minnows exhibited highly variable survival at the four test sites. Spottail shiner tested only during the spring, which were in poor condition when received, yielded values ranging from 36.4% at Five Channels to 73.5% at Rogers. Alcona survival was intermediate between the two at 59.5%. A golden/common shiner mix tested during the fall at all four locations exhibited relatively high survival, averaging 82-85%, except for juveniles at Rogers, which had a survival rate of 53.7%. This low survival is in direct contrast to the Rogers adult survival, 92.5%, the highest value recorded for the group.

Walleye and yellow perch exhibited the same general trend in survival among the four test sites, with lowest survival at Alcona, intermediate survival at Five Channels, and highest survival at Rogers. Hardy had intermediate survival for walleye and high survival for yellow perch. Field observations indicated that the walleye and yellow perch tested at Alcona - but not at the other test sites - suffered high mortality during shipping and the 24-h period prior to testing. The results of the Alcona tests were probably biased on the low side due to the highly stressed nature of the fish.

Survival results obtained for rainbow trout were highly variable, ranging from 71.4% at Hardy to 100% at Alcona for juveniles, and from 61.2% at Rogers to 89.4% at Alcona for adults. Overall, Alcona had the lowest impact on rainbow trout, with survival averaging 95%. Hardy had thegreatest impact on rainbow trout, with an average survival value of 70%. Mortality was highest for adults, primarily because of cuts and abrasions.

Adult northern pike had high survival at Five Channels (91.3%) and Rogers (83.4%), intermediate survival at Hardy (76.0%), and relatively low survival at Alcona (51.2%).

White sucker survival was generally high at all locations, with adult survival lower than juvenile survival. Alcona had the highest white sucker survival (approximately 92% for both life stages), Hardy the lowest, averaging 70.7%.

Analysis of turbine entrainment survival data using FIRM is presented in Figure 1. For turbine-introduced fish the greatest influence on survival was total length (age), with very small fish (≤50 mm) and the larger fish (>300 mm) having the lowest survival. For fish in the 51-300 mm length range, which accounted for the greatest percentage of the fish tested (76.1%), a significant species influence was determined. Rainbow trout, bluegill, northern pike, yellow perch, and spottail shiner had the highest survival, 88.6%; walleye, golden shiner, and white sucker had intermediate survival, 80.8%; black crappie and largemouth bass had the lowest survival, 68.0%. Overall, the different physical and operational conditions noted among the four projects had only a minor influence on turbine entrainment survival.

Program Summary and Conclusions

Consumers Power Company sponsored biological monitoring studies at 11 hydroelectric projects in Michigan as part of Federal Energy Regulatory Commission (FERC) relicensing activities. Six of the 11 projects are located on the Au Sable River; the other five are in western Michigan, two on the Manistee and three on the Muskegon. Based on physical and operational characteristics, the 11 projects were separated into four groups. One from each group was selected for turbine entrainment survival testing: Hardy and Rogers on the Muskegon River and Five Channels and Alcona on the Au Sable River.

Turbine entrainment survival testing was accomplished by introducing marked fish directly to the turbine as close to the wicket gates as possible and collecting them in a floating live car attached to the cod end of a draft-tube net that intercepted the entire turbine flow. The regulatory agencies selected the fish species used in the study to represent those resident in the three river systems and that are susceptible to entrainment. In addition to turbine-released fish, two control groups were monitored, one to evaluate

stress related to fin clipping and holding and the other to monitor collection (net effect) and handling stress.

The fish release and collection technique employed was very effective: 84.5% of live fish and 92.6% of dead fish released at the turbine were collected in the draft-tube net. Except for strong-swimming fish species like rainbow trout and white sucker, which had collection efficiencies of 59.7 and 83.5%, respectively, fish were generally collected within a few minutes of release. Some fish were observed swimming in the draft tube and the mouth area of the draft-tube collection net following release.

Turbine entrainment survival averaged 83.5%; variability among the four projects was slight, ranging from a low of 79.1% at Hardy to 86.0% at Five Channels. The Hardy project differed from the other three by having two separate discharge bays per unit, with unequal discharge flow noted between the bays. These separate discharge bays, coupled with high velocities and turbulence, limited sampling to one seasonal period (fall), and may have contributed to the lower overall collection efficiency and mortality observed there.

Peripheral runner velocity has been a major concern in relation to turbine passage survival for Francis-type turbines (EPRI 1987). Of the four projects evaluated, Hardy has the highest peripheral runner velocity, which may account for the lower overall survival determined for this project. The difference in project design and operational characteristics had no significant influence on survival; the primary influence was the size (age) of the fish, with the very small and largest length categories having slightly lower survival. Fish in the 50- to 300-mm range had the highest survival. The incidence of blade strikes to large fish at the four projects was not observed for smaller fish. Seasonal difference in entrainment survival, which translates primarily to different ambient water temperature, also had no substantial influence on survival.

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Survival of Warm-water Fishes In Turbine Passage at a Low-head Hydroelectric Project

Dilip Mathur, Paul G. Heisey and Douglas D. Royer[1]

Abstract

The self-inflating HI-Z Turb'N Tag-recapture technique (U.S. Patent No. 4,970,988) was utilized to estimate survival of channel catfish (Ictalurus punctatus) and bluegill (Lepomis macrochirus) in passage through a low-head hydro project in North Carolina. Tests were conducted on two age groups at two turbine runner blade settings (13^{o} and 28^{o}). The short-term survival (1 hr) of young (< 220 mm Fork Length) and adult (> 220 mm FL) channel catfish under a worst case operating condition (blades set at 13^{o} angle) was 90% and 79%, respectively. The long-term survival (48 h) of both size groups was almost 100%. The short-term turbine survival for both age groups at near normal operating conditions (turbine blades at 28^{o} angle) was 93% and long-term survival was 100%. For young bluegill, the 1 h survival rate was 96% and for adults it was 86%. The 48 h survival was 99% and 100% for juvenile and adult bluegill, respectively. Results from this and other similar studies indicate that survival of

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[1]RMC Environmental Services, Inc., Utility Consulting Division, 1921 River Road, P. O. Box 10, Drumore, Pennsylvania 17518

fishes in passage through propeller type turbines is much higher than reported in some previous studies and is a function of fish size and shape, turbine blade runner speed, and blade angle.

Introduction

The recent surge in developing potential hydroelectric sites (old, abandoned, or new construction) and relicensing of existing projects has spurred concerns for turbine-related fish mortality. Resource and regulatory agencies are concerned that these projects inflict severe damage on fish populations. As a result, turbine entrainment/mortality studies are undertaken essentially at all hydro projects and expensive mitigation or corrective measures (e.g., bypass systems, screening, and behavioral devices) are being recommended. However, in our view, reliable data are often lacking to support such recommendations particularly for nonmigratory resident fishes. Much of the work on turbine-related mortality has focused on migratory fishes, particularly salmonids and alosids (Ruggles et al. 1990; Heisey et al. 1992a,b; Mathur and Heisey 1992).

Ruggles and Palmeter (1989) reported that total fish recapture is the most effective method to quickly gather unambiguous results for estimating turbine-related mortality. However, logistical problems encountered using other methodologies (e.g., netting and radio tagging) often preclude physical recapture of a high proportion of fish and seriously impede obtaining a clear insight into the magnitude and nature of turbine passage mortality. In these cases other methods are needed to obtain reliable information on short and long-term effects of turbine passage on fishes. Netting techniques often inflict high control mortality if, for example, nets are not equipped with a live car. A high control mortality can frequently result in high estimates of turbine-related mortality (Heisey et al. 1992b).

The objectives of this paper are to present (1) application of the self-inflatable HI-Z Turb'N tag (U.S. Patent No. 4,970,988) to determine survival of channel catfish (*Ictalurus punctatus*) and bluegill (*Lepomis macrochirus*) in passage through turbines at a low-head hydroelectric project and (2) effects of two turbine operational conditions on short-term and long-term turbine passage survival. Earlier reviews (Bell 1981; Eicher 1987) had indicated that survival of fishes is higher at efficient turbine operational modes.

Project Description

Craggy Dam Hydroelectric Project is situated on the French Broad River in Asheville, North Carolina. The project facilities are approximately 600 m above sea level and extend from river kilometer 226 to 228.

The Craggy Dam, built in 1904, is the uppermost dam on the river. It was operated as a run-of-the-river hydroelectric station until 1963 when it was abandoned by the Carolina Power and Light Company. Metropolitan Sewerage District of Buncombe County (MSD) renovated the facility and began generating electricity in spring of 1987 (Woodruff 1987). The power station diverts water from the river for a total distance of 975 m via a 853 m long intake flume to the powerhouse and 122 m tailrace. Water discharged by the power station re-enters the river channel approximately 975 m downstream from the dam. A seasonally adjusted Federal Energy Regulatory Commission approved minimum flow is maintained in the bypass reach.

The 2.1 MW power station consists of three S-type bulb turbines, each rated at a discharge of 18 m^3/s. Unit No. 2 is a variable flow unit and its turbine runner blades can be adjusted from 0^o to 34^o angle settings. Blade settings are normally kept between 10^o (inefficient) and 28^o (efficient) angles for power generation. Blade-to-blade clearance increases with increased angle settings. The other two units have fixed 34^o angle blades. Other specific hydraulic characteristics are as follows: net head 6 m, number of blades 4, runner speed 229 rpm, runner diameter 175 cm, clearance between blades at hub 30 to 33 cm, and clearance at tip of blades 41 to 46 cm.

Methods

Fish passage survival tests were conducted at Unit No. 2 when it was operating at normal inefficient (13^o) and efficient (28^o) turbine blade settings. Channel catfish and bluegill were obtained from the Cape Fear Fish Farm, North Carolina, and were held in an aerated circular 2,000 L tank. Tanks were continuously supplied with river water via a submersible pump. The water temperature ranged from 20.5^oC to 23.5^oC. Channel catfish < 220 mm FL were considered juveniles; those larger were considered adults. Bluegill < 120 mm FL were considered juveniles and those larger were considered adults.

Fish were individually netted from the tanks and measured. Each fish was fitted with one to three HI-Z

Turb'N Tag(s). Additionally, a semi-buoyant miniature radio tag was attached to most specimens (Figure 1). Feasibility of using this tag-recovery technique, and its effects on the behavior of fish had been determined in the laboratory prior to field tests. Both the radio tag and Turb'N Tag were attached to fish using a modified ear piercing gun (Heisey et al. 1992). Attachment of both tag types allowed easy recovery by monitoring the radio signal and then visually locating the Turb'N Tag(s) upon inflation.

Turb'N Tags were made of brightly colored latex and were pear shaped with an approximate length and width of 38 mm and 13 mm, respectively. Each Turb'N Tag weighed 1.5 g. Upon inflation the tags measured approximately 75 mm long and 50 mm in diameter. Each radio tag was cylindrical and approximately 10 x 31 mm, weighed 1.7 g, and propagated radio signals through a 27 cm thin wire antenna. The number of Turb'N Tags attached to a fish was dependent upon size and swimming stamina. Small fish usually received one Turb'N Tag while larger fish received three Turb'N Tags. Each Turb'N Tag was attached by a stainless steel pin (5 or 10 mm long) through the musculature of the fish near caudal, anal and/or pectoral fins. Each pin was secured by a small plastic disc. The radio tag was attached in combination with one of the Turb'N Tags by a single pin so that one tag trailed on either side of the fish.

Tagged test fish were introduced individually into the turbine penstock by an induction apparatus consisting of a small holding basin (approximately 75 L) attached to a 10.2 cm supply/delivery/line (Heisey et al. 1992). A gasoline-powered trash pump supplied water to the system to ensure fish were transported quickly within a continuous flow of water through the reinforced plastic delivery line. Control fish were tagged and released individually through the same induction apparatus directly into the discharge "boil" of the operating test turbine unit to separate the effects of turbine passage from that due to tagging/handling and induction. Time from release to recapture was recorded for each fish.

Test and control fish were recaptured from the tailrace, placed in an on-board holding tank, examined for mechanical injury, and the tags and pin(s) were removed. Recaptured fish were initially transferred to a 2,000 L circular tank on shore. For long-term survival assessment (48 h) the fish were retained in a 2,000 L tank or floating net pen. Net pens were 1.5 m in diameter, 1.2 m deep, and covered with 7 mm rigid plastic netting. Radio tags recovered from each fish

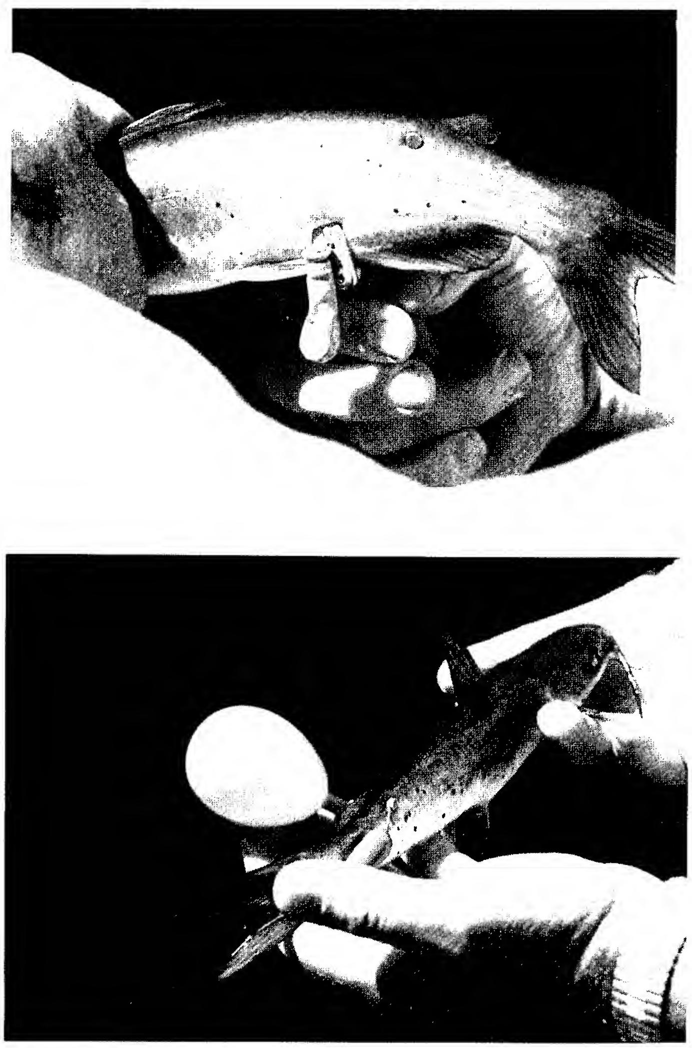

Figure 1. Uninflated and inflated Turb'N Tag.

were reused on subsequent specimens until the tag battery expired.

Recaptured fish were categorized as follows: (1) short-term survival if fish were alive for 1 h, (2) noncaptured fish with radio signals indicating it was active was also classified short-term survival, (3) unknown, if the status of any fish could not be positively determined within 30 min by direct observation or the radio signal pattern. Recaptured live fish were held for assessing long-term mortality (48 h).

The 1 h and 48 h survival rates of fishes passing through the turbine were estimated using the formula given in Burnham et al. (1987):

$$S = \frac{\text{Proportion of live test group}}{\text{Proportion of live control group}}$$

Results

Recapture Rate

The recapture rate of fish after passing through an operating turbine was high (Table 1). Overall, 90% and 100% of the test and control channel catfish, respectively, were physically recovered.

TABLE 1. Recapture rates and estimated survival of channel catfish and bluegill at two turbine runner blade angle settings (13° and 28°) at the Craggy Dam Hydroelectric Station, August-September, 1990).

Species/Size	Turbine Setting	No. Released		Recapture Rate		1 h Survival	48 h Survival
		Test	Control	Test	Control		
Channel catfish (< 220 mm)	13°	63	28	0.90	1.00	0.90	0.98
Channel catfish (< 220 mm)	28°	43	28	0.93	1.00	0.93	1.00
Channel catfish (≥ 220 mm)	13°	39	22	0.90	1.00	0.81	1.00
Channel catfish (≥ 220 mm)	28°	32	22	0.88	1.00	0.93	1.00
Bluegill (< 120 mm)	13°	33	40	0.85	0.90	0.96	0.99
Bluegill (≥ 120 mm)	13°	72	54	0.90	0.96	0.86	1.00

The status of another 4% of the test fish was discernible. Recapture rates at the two turbine blade settings were almost identical ($P > 0.05$). No differences ($P > 0.05$) were noted between test and control fish recapture rates. The overall recapture rates of test and control bluegill were 88 and 94%, respectively. No differences ($P > 0.05$) were noted between recapture of test and control groups.

1 h Survival of Channel Catfish

A total of 102 channel catfish of two age groups was passed through Unit No. 2 at a turbine blade setting of 13°. Additionally, 75 fish were introduced when the blade setting was at a 28° angle.

Differences in survival between turbine blade settings were minor and statistically nonsignificant ($P > 0.05$). Overall, the 1 h survival of two age groups was 86% at the 13° turbine blade setting. At a 28° blade angle, survival of both juveniles and adults was 93% (Table 1).

48 h Survival of Channel Catfish

All live test (148) and control (50) fish recaptured were placed into net pens to assess delayed effects of turbine passage at both turbine blade settings. No controls died. Turbine blade settings did not affect long-term survival of fish. The 48 h adult survival was 100% at both turbine settings. Some 98% of test juveniles were alive after 48 h at the 13° turbine setting and 100% at the 28° setting.

1 h Survival of Bluegill

A total of 40 young and 65 adult bluegill was tested at a worst case operating condition (13 °). Similar numbers of control fish were also released. None of the control specimens suffered short-term mortality. The overall short-term survival was 89% for young and adult bluegill (Table 1).

Slight differences in survival of the two age groups of bluegill occurred (Table 1). Young bluegill showed a survival of 96% while adults showed a survival of 86%. Though bluegills were not tested at a higher turbine blade setting (28°) it is expected that survival would be higher due to greater blade clearance.

48 h Survival of Bluegill

Eighty-four test and 88 control fish were recaptured alive and held in net pens for long-term observation. The overall survival of test fish after 48 h was 79.8% and 80.7% for controls. The survival rate of juvenile test fish was slightly lower than in the control group while the opposite was true for the adults (Table 1).

Discussion

The HI-Z Turb'N Tag and recovery procedure performed well. Of the 227 fish tagged and released the status was known for approximately 95% of them. The high recapture rate in combination with high short-term survival allowed for reliable assessment of long-term survival, particularly for channel catfish. Since most fish were physically recaptured it allowed for examination of each fish for the type, extent, and location of turbine induced injury. Additionally, since the Turb'N Tag-recovery technique did not inflict any injuries on the control specimens, the direct effects of turbine passage were readily discernible with relatively few assumptions. We have observed similar results on other species at other hydro projects (Heisey et al. 1992a,b; Mathur and Heisey 1992; RMC 1992).

As expected, some size-related differences in survival were noted. However, these differences were < 10%. The survival of young bluegill (< 120 mm) was 96% while that of larger sized fish was 86%. Although the survival of channel catfish was not size related at a 28^{o} turbine blade angle, there was a difference (9%) in survival between large and small size groups at the 13^{o} turbine blade angle.

Most fish that died or were injured showed body cuts and lacerations due to contact with turbine blades. However, the direct contact of fishes with the turbine blades appears to be a low probabilistic event particularly at a higher blade angle setting (28^{o}, peak turbine efficiency). Under peak turbine operating efficiency, the probability of direct contact of fish with the turbine blades is reduced because of the wider clearances between turbine runner blades. The turbine unit tested for this study is not normally operated at a low blade angle setting. Ruggles (1980) concluded that survival of fishes was highest at the point of highest turbine operating efficiency.

The high survival rates of resident fishes estimated in the present study are similar to those recently reported at other low-head projects (< 30 m). At the Chalk Hill Project (Kaplan turbine), a 7.8 MW station on the Menominee River, Wisconsin the immediate survival of bluegill was estimated at 97%; the survival of white sucker/rainbow trout was estimated at 93.4% (RMC 1992). The latter species are shaped similarly to channel catfish tested in the present study. Little mortality occurred after 1 h. General conclusions that may be drawn from the present study and the above cited study are that survival of resident fishes, particularly those < 200 mm long exiting low-head projects equipped with propeller type turbines rotating at slow speeds, is much higher than reported in some earlier studies (EPRI 1992). If the fish survive immediate turbine passage there is a low probability of delayed mortality (48 h).

Additional evidence indicates that the actual turbine passage survival of resident species may be higher under natural conditions than observed during the study. Test conditions in the present study involved a forced induction of fishes into the penstock; it is unlikely that many resident fish species would voluntarily move through the turbines (Cada 1990). Warm-water resident species generally do not normally undertake voluntary long downstream movements. This behavioral characteristic reduces the risk of turbine entrainment and subsequent mortality. Ruggles et al. (1990) reported that mortality of naturally entrained fishes (alewife, Alosa pseudoharengus; Atlantic salmon smolts, Salmo salar and yellow perch, Perca flavescens) was considerably lower than those force-fed into penstocks.

Acknowledgements

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Hydroelectric Licensing: Toward a New Federalism

Peter W. Brown[1]
Daniel W. Allegretti[2]

Abstract

This paper will explore areas in which state and federal regulation tend to deviate. This paper will also identify issues that must or should be addressed in order to harmonize federal-state relationships. The authors have also selected a model of a regulatory regime in order to examine a structure in which federal-state relationships may be harmonized. The model is based on the assumption that the Federal Power Act could be amended to vest regulatory authority over hydropower in the Federal Energy Regulatory Commission ("FERC") but at the same time empower FERC to delegate implementation of a portion of its authority to state agencies. Such implementation would be pursuant to plans proposed by state agencies and approved by the FERC. In discussing such a model, the authors of this paper do not take a position that such a model is the only or most desirable structure or, even, is appropriate for implementation at any time in the future. Rather, it is their intent to focus the reader's consideration on the need to examine alternatives to the current licensing scheme in order to better address state and federal licensing interests.

[1] President, ELI Corporation, 501 South Street, Concord, NH 03304

[2] Senior Research Associate, ELI Corporation, 501 South Street, Concord, NH 03304

The Current Regulatory Regime

Beginning with the enactment of the Federal Water Power Act in 1920, water has been viewed as a public resource and hence subject to licensing. Under that same law the United States has assumed an expansive responsibility for licensing hydroelectric facilities. The Federal Energy Regulatory Commission has jurisdiction over hydroelectric plants of every scale and configuration located throughout the country. Indeed, examining decisions on the scope of FERC jurisdiction reveals that the trend in recent years has been toward a broader interpretation of its jurisdiction. See Habersham Mills v. FERC, 976 F.2d 1381 (11th Cir. 1992) (Small projects not even selling power were required to obtain licenses because of their cumulative effect with similar projects on interstate commerce); Clifton Power Corporation, 39 FERC ¶ 61,177 (1987) (FERC gave broad interpretation to its authority to issue licenses to projects on "commerce clause" waters). But see Guy M. Carlson, 62 FERC ¶ 61,009 (1993) (In a 3-2 decision FERC found that the mere presence of anadromous fish in a waterway which would be impacted by a project was not a basis for asserting jurisdiction).

At the same time, however, we have witnessed in the last decade and a half the enactment of three major laws amending the Federal Power Act and redefining the federal role in hydroelectric plant licensing: the Public Utility Regulatory Policies Act of 1978 (PURPA), the Electric Consumers Protection Act of 1986 (ECPA), and the Energy Policy Act of 1992 (EPACT).

PURPA established simplified and expeditious licensing procedures for small hydroelectric projects (5000 kw or less) and authorized FERC to exempt such projects from licensing requirements in certain cases. Such exemptions, however, must include conditions that the U.S. Fish and Wildlife Service, National Marine Fisheries Service and state fisheries agencies require.

ECPA substantially increased the role and influence of states under Part I of the Federal Power Act. In the wake of ECPA, FERC was directed to consider state interests under any "comprehensive plan" for a waterway developed by a federal or state agency. ECPA also directed FERC to give equal consideration to environmental values in deciding whether to issue a license.

EPACT, among other hydropower provisions, limited the eminent domain authority under the Federal Power Act to allow state and local governments to retain control over public property that might otherwise have been condemned under a license.

The result of all this is an increasingly complex and balkanized system that results in a difficult role for FERC and leaves many state agencies dissatisfied with their involvement in the process. The FERC is by no means the sole or even the ultimate authority in setting a policy for hydropower and licensing projects. Yet FERC remains at the center of the process and continues to issue comprehensive federal licenses for virtually all hydroelectric plants. The question that is suggested is why do we continue to have comprehensive federal licensing for hydropower in every case? No other technology available for generating electricity, except nuclear fission, must in every case be licensed by a federal agency in a comprehensive way. If the differences between hydropower and nuclear power are not obvious, they should be.

All this has led the authors to suggest the concept of a new federalism.

Alternatives to the Current System: -- Toward a New Federalism

The concept of a new federalism is simply the elimination of comprehensive federal licensing for all hydroelectric plants. The alternative to this status quo is obvious: less federal regulation and more state responsibility. Several models for this suggest themselves.

First, the Federal Power Act could be amended so as to modify FERC jurisdiction. Jurisdiction could be narrowed based on some criterion or combination of criteria such as project size, degree of environmental impact, etc. (Brown, et al. 1993; DOE, et al. 1990) In the absence of FERC preemption over such projects, state and local authorities would presumably step in and regulate. In some respects the current exemption status for conduit projects and projects 5 MWs or less at existing dams embodies this approach.

Second, Congress could expressly designate some categories of projects as subject to FERC licensing and

others as subject to state licensing under a federal mandate. Such a mandate might carry with it standards and limitations to be observed by state licensing authorities.

Finally, Congress could vest jurisdiction in FERC but allow FERC to delegate licensing authority to individual states by approving state implementation plans. Such plans are, for example, used to implement the Clean Air Act. Under this model, a state would be free to refrain from putting forward a plan, in which case FERC would license projects within the state borders. Moreover, in the case where a plan is put forward by a state, FERC would retain control in protecting federal interests by virtue of its authority to approve or reject the plan. By assigning a principal, supervisory role to FERC in the development of such a system, participants in the licensing process, including the states themselves, could take advantage of the institutional knowledge of the FERC and benefit from the considerable experience FERC has in licensing.

These are merely some, and by no means all, of the possible models for a new federalism. Selecting among the alternatives is itself an important policy question. Any analysis, however, must start somewhere and therefore the authors wish to posit the latter model of state implementation plans as a starting point for discussion.

<u>Critical Issues in the Development of a New Regulatory Model</u>

As noted above, the authors posit a model of a State Implementation Plan ("SIP Model") primarily because it is a useful analytical tool to comprehend the complex web of interests and institutional arrangements embodied in the current regulatory system. States and the federal government, moreover, have extensive experience with the SIP Model in connection with enforcement of the federal Clean Air and Clean Water Acts, among other regulatory regimes. Hence, there should not be a great deal of discomfort with notions such as state options to undertake a SIP, minimum requirements and guidelines for SIPs and FERC review and approval of SIPs. As noted above also, such a model draws upon the considerable expertise of the Office of Hydropower Licensing at FERC in the full gamut of issues ranging from river basin studies to enforcement.

With respect to the SIP Model as an analytical tool, the model forces an identification of the important federal interests that must be protected. In this regard, it is necessary to review Part I of the Federal Power Act and the amendments to it under PURPA, ECPA and EPACT in order to determine what those interests are and their relative importance.

Certainly, federal protection of national or regional environmental and natural resources is a paramount interest to be protected by the FERC with the considerable participation of sister federal resource agencies. Given the relative importance of this interest, should the FERC retain ultimate decision-making authority on these issues on a case-by-case basis? Or, can the decision-making process for categories of cases be delegated to the states, subject to standards carefully drawn by the FERC based on its considerable knowledge and experience?

A second important federal interest is hydroelectric development at federally owned facilities (e.g. Corps and Burec dams) and on federally owned lands. Should states be allowed to authorize private hydroelectric development at federal facilities? In this regard the recent statutory repeal of the Henwood decision by EPACT provides a clear signal from Congress that federal land management agencies should retain the ultimate and final authority with respect to hydroelectric development on federal land -- even over the decisions of their sister federal agency, the FERC.

There are other uniquely federal interests embodied in the Federal Power Act and amendments, but there is a significant question as to their current importance. The "Anti Site Banking" provisions were borne of the fears, early in this century, that unregulated monopolies would acquire and prevent development of the best sites and foreclose electric consumers from the benefits of a significant source of electric power. Is there sufficient federal interest now in the site banking provisions of the FPA to require SIPs, in order to be approved, to adopt such provisions?

With respect to state interests, in order for the SIP Model to work, it is critically important to identify the state interests embodied in the FPA and amendments. It would be folly, indeed, to mandate a SIP and not permit the implementing state agency to protect the state

interests recognized in the FPA and its amendments. For example, the implementing state agency must be authorized to assure that a hydroelectric project is consistent with a state comprehensive plan for the particular river basin. In this regard, and given the historic place of similar provisions in other federal legislation, it would seem appropriate that SIPs be permitted to recognize the "municipal preference" as embodied in the FPA and FERC and judicial decisions on the subject.

As with all SIPs under the Clean Air and Clean Water Acts, the regulatory system embodied in SIPs must have the requisite resources and be workable. Provisions in the FPA dealing with the licensing fees, enforcement, dam safety inspection and reporting requirements are all features of any regulatory regime that is workable; -- there is no particular federal or state interest implicated in these provisions. Accordingly, it may not be necessary to require a SIP to adopt these provisions whole cloth (e.g. the level of licensing fees) in order for that SIP to be approved. So long as the requisite resources are devoted to the effort and there are enforcement mechanisms and fiscal support a SIP should meet the standards. Again, states developing a SIP should draw upon the successful experience of FERC in managing its resources to perform these types of functions and tailor their own program accordingly.

Finally, in any change as contemplated by the SIP Model for hydroelectric regulation, there must be transition rules. Here the issues are wide ranging and serious. What happens to existing licenses? Pending applications? What is the timing for approval of a SIP? If because of the transition rules, a state will not have to review hydroelectric applications until sometime in the future, can the state wait before it seeks approval of a SIP?

Conclusion

The foregoing has outlined some of the issues associated with the development of a new regulatory system along the lines suggested by the SIP Model. Obviously, many more issues need to be assessed and discussed before serious consideration can be given to the SIP Model or any other alternative to the current system. This paper hopes to begin this assessment and discussion.

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DEVELOPMENT OF A MATHEMATICAL MODEL FOR PREDICTION OF SHAD POPULATION GROWTH ON THE SUSQUEHANNA RIVER

Michael F. Dumont[1] and Peter S. Foote[2]

ABSTRACT

Since 1970 an active program has been undertaken to restore American shad to the Susquehanna River Basin. This program has been funded by the owners of the four hydroelectric facilities on the lower river (Conowingo, Holtwood, Safe Harbor and York Haven), and has been administered in cooperation with state and Federal resource agencies, under the direction of the Susquehanna River Anadromous Fish Restoration Committee, (SRAFRC).

The following specific activities have been implemented:

- Construction of two fish lifts at Conowingo, with collection and handling facilities for trucking the fish to the spawning areas above York Haven;
- Construction and operation of a shad hatchery;
- Closure of the sport and commercial fisheries for shad in the upper Chesapeake Bay and the lower Susquehanna River; and
- Various biological and fish behavioral studies relating to upstream and downstream fish migration in the river.

This paper describes a mathematical model that has been developed to investigate the future growth potential of American shad in the Susquehanna River for two alternate fish passage development scenarios; and to review the most appropriate schedule for any proposed new construction.

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[1]Project Manager, and [2]Environmental Department Head, Acres International Corporation, 140 John James Audubon Parkway, Amherst NY, 14228-1180

INTRODUCTION

In 1989 the U.S. Fish & Wildlife Service issued a memorandum giving details of proposed target design populations for three species of anadromous fishes in the Susquehanna River, and criteria to be used in the design of fish passage at the three upper dams (St. Pierre, 1989). The target design populations from this letter are summarized in the following table:

SUMMARY OF U.S. FISH AND WILDLIFE SERVICE DESIGN CRITERIA

	Holtwood	Safe Harbor	York Haven
Project Description			
No. of units	10	12	20
Hydraulic capacity (m^3/s)	906	3,110	450
Installed capacity (MW)	108	417	20
Target Design Population (Millions)			
American shad	2.7	2.5	2.0
Blueback herring	5.0	2.5	0.5
Alewife	5.0	2.5	0.5

In 1990 Acres International Corporation was retained by the utilities to perform a feasibility study on the proposed construction of fish passage at the three stations upstream of Conowingo. The tasks required included an assessment of the current rate of growth of the existing American shad population; development of conceptual fish passage designs at the three dams; and a comparison of the cost and effectiveness of this multiple development with the alternative of continued trapping and trucking of fish from Conowingo to the river above York Haven.

The future rate of growth of the American shad population is dependent on the complex interaction of a variety of factors, including the biology and behavior of the fish; the spawning areas available in the lower river reservoirs and the upper river basin; the efficiency of existing and future proposed fish passage designs; the extent of fish mortality during migration; the operation and possible future closure of the shad hatchery; and the future opening of the sport and commercial fishery.

The mathematical model presented here was developed in order to evaluate and if possible mitigate the impact of

these various factors on the population growth of American shad in the Susquehanna River. The model was also used to evaluate the schedule for construction of fish passage at the three dams above Conowingo.

DESCRIPTION OF THE MODEL

The mathematical model is a spreadsheet algorithm of the spawning of American shad on the Susquehanna River. Each line of the spreadsheet represents one yearly spawning cycle.

The basic assumption of the model is that the parent stock of fish for the restoration program are those adult shad that ascend the Susquehanna River to the Conowingo Dam tailrace each year. The parent stock at the beginning of each yearly cycle is calculated as the sum of the returning adults from previous years spawning, (Year Classes II through VII), with an additional small annual contribution from hatchery releases.

The average ratio of future progeny (recruits) from the parent stock (the "stock/recruitment ratio") is assumed to be 2.0, as determined from data contained in the SRAFRC annual reports from 1980 through 1989. The age distribution of the recruits in future years has been assumed to be similar to the Susquehanna River age distribution over the same period, 1980-1989.

It is also assumed that the parent stock spawns successfully each year, until such time that the capacity of the available spawning habitat is reached. At this point the population will reach its maximum value within about five years; any additional recruits will be lost to the system.

For the trap and truck program from Conowingo Dam, adult shad have the opportunity to spawn in three locations: the Conowingo tailrace (or lower river), Conowingo reservoir (if passed directly over the dam), or the upper Susquehanna River (above York Haven), after release from the trucks.

When fish passage facilities are constructed at the upper three dams, the spawning areas available will include: the Conowingo tailrace (or lower river), the Conowingo reservoir, the Holtwood reservoir, the Safe Harbor reservoir and the river reach up to York Haven, and the river upstream of York Haven.

The maximum capacity of each segment of the river for spawning was determined by estimating the area of suitable spawning habitat in each river segment, and multiplying by

the number of spawning shad per unit area as determined in previous studies on the Connecticut River, (St. Pierre, 1979; Crecco and Savoy, 1987).

The number of fish that will reach the various spawning areas will depend on the efficiency of the various fish passage facilities, and the trucking program. For the purposes of the mathematical model, fish passage efficiency is defined as the percentage of fish arriving at a dam that successfully locate and pass through the facility into the reservoir above. For the present studies, the fish passage efficiencies are assumed to be the same for each project. A range from 50 to 90 percent was examined, based on the range of estimated efficiencies at other east coast fish passage facilities.

For the scenario of continued trapping and trucking from the Conowingo Dam to the reach above York Haven, the fish mortality due to the stress of trucking was assumed to be 5 percent, based on data from Philadelphia Electric Company (PECO), the owner of Conowingo. The maximum number of trucks was based on a fish lift loading cycle time of 15 minutes at Conowingo, and a round trip of four hours from Conowingo to the discharge point above York Haven, giving a peak day usage of 16 trucks for each of the Conowingo fish passage facilities.

The contribution of the existing hatchery operation was factored into the model by estimating the number of females that would be required in the wild to produce the number of fry and juveniles released by the hatchery. For 1989, this was determined to be about 7,000 females. To estimate the total hatchery "parent stock" the number of females was multiplied by 3 (based on normal sex ratio of 2 males: 1 female). The hatchery contribution, 21,000 fish, was assumed constant throughout the analysis.

Appendix A gives a detailed summary of the development of the model, including a listing of the main assumptions; and the methodology used in determining the available parent stock in each year and the returning progeny in future years.

MODEL RESULTS

The mathematical simulation model was used primarily to compare the predicted performance of the multiple fish passage option with the alternative option of trap and transport from Conowingo, using a variation in fish passage efficiency from 50 percent to 90 percent.

Estimated long-term population growth potential for each option is summarized in the following table:

ESTIMATED POPULATION IN YEAR 2040 (MILLIONS)

Fishery	**Trap & Truck**	**Multiple Fish Passage**				
		Fish passage efficiency:				
		50%	60%	70%	80%	90%
None	1.69	1.48	1.96	3.38	8.03	8.03
20% Harvest	1.35	1.11	1.36	1.94	3.66	6.43

A typical graphical representation of the projected population growth to year 2040 is shown on Figure 1, for the case where fish passage facilities are constructed at the upper three dams in 1996, and the fish passage efficiency is 70 percent.

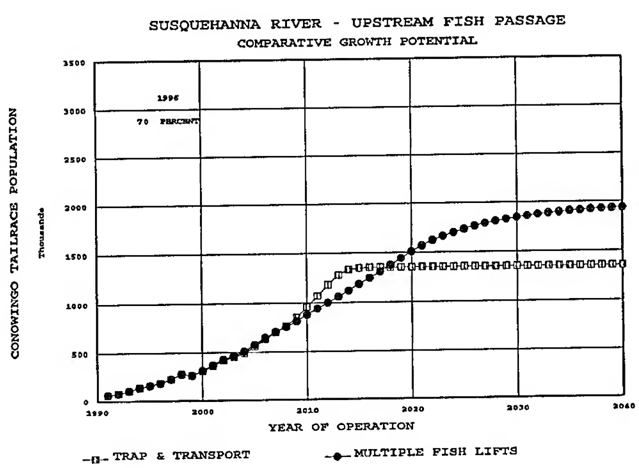

Figure 1 - Comparative Growth Potential

The following conclusions can be drawn from the model when comparing alternative fish passage development options:

(1) The multiple fish facility option is superior to the trap and transport program from Conowingo Dam in achieving an ultimate population size, if each fish facility has an efficiency of at least 60 percent;

(2) The USFWS population goals can be met at each of the four dams with the multiple fish passage option, if the efficiency of each facility exceeds 80 percent;

(3) Trapping and trucking of fish from Conowingo is not a long-term option that will achieve USFWS population goals at any of the four dams; and

(4) Trapping and trucking of fish from Conowingo may, however, be a viable short-term option, since population growth comparable to the multiple fish facility option may be maintained until the maximum handling capacity of the trucking program is reached.

SENSITIVITY ANALYSIS

The results of the mathematical simulation have been analyzed for variation in the major design parameters and the major assumptions made, in order to assess the sensitivity of the conclusions drawn.

The following is a summary of the results of the sensitivity analysis:

(1) The population growth potential predicted by the model at each site for the multiple fish passage option is not sensitive to the year of construction, provided the facilities are operational before the peak capacity of the trap and transport operation is reached.

(2) The model results are significantly impacted by the assumed stock-recruitment ratio. If the ratio is reduced from 2.0 to 1.5, then the population goals cannot be achieved, regardless of the efficiency of the fish facilities at each site. If the ratio is increased from 2.0 to 2.5, then the population goals are achieved up to 10 years earlier.

(3) The multiple fish passage option is still superior to the option of trap and transport from Conowingo, in achieving an ultimate population size, with any of the alternative stock-recruitment ratios tested, provided the fish passage efficiencies exceed 60 percent.

(4) The results of the analysis are not changed numerically if the hatchery operation is discontinued, but the year of achievement of the population goals is delayed by four years.

(5) The results of the analysis are significantly impacted by spawning of shad in the intermediate reservoirs before they reach the next upstream dam. This is because of the limited spawning capacity available in the lower reservoirs as compared with the river basin above York Haven (see Appendix A, Section A.2).

If 10 percent of the upstream-migrating fish spawn in each reservoir, this is equivalent to a loss of 7 percent in fish passage efficiency, with a corresponding significant reduction in long-term population growth.

If 20 percent of upstream-migrating fish spawn in each reservoir, the target population goals at each site cannot be reached, regardless of the efficiency of the fish passage facilities.

CALIBRATION AND VERIFICATION

From a review of the sensitivity analysis, the most significant parameter in estimating future trends in population is the stock/recruitment ratio. For the comparison of development options, a constant value of 2.0 was used. Based on data from the Connecticut River (Crecco and Savoy, 1987), this ratio may vary from as low as 0.7 to as high as 3.5 in any year, depending in large part on river flows and temperatures during the spawning period.

The likely accuracy of the model, using a variable stock/recruitment ratio, was assessed by predicting the Conowingo tailrace population from 1984 through 1990, and comparing the results with the tailrace population estimates made by Maryland DNR from 1984 through 1990. Available Susquehanna River data from 1980 through 1985 were used to estimate the parent stock. The stock/recruitment ratios from 1980 through 1985 were calculated from SRAFRC data. From 1986 onward, variable stock/recruitment ratios, calculated from Connecticut River data for the years 1966 through 1980, were utilized.

Figure 2 compares the output from the model with the Maryland DNR population estimates from 1984 through 1990. The correlation between the estimated American shad tailrace population from the model, and the actual Maryland DNR tailrace population estimates is excellent.

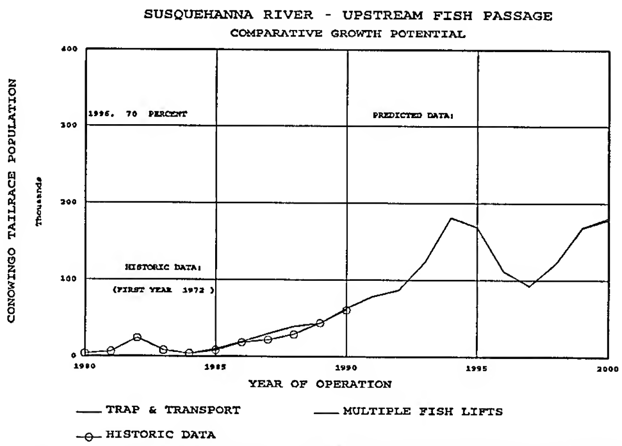

Figure 2 - Comparison of Predicted Populations With Maryland DNR Population Estimates

CONCLUSIONS

The mathematical model described is a useful planning tool for evaluating options available for multiple fish passage development, and for determining the effects of several different factors on the growth of the shad population in the Susquehanna River. As with any model, a number of assumptions have been made; however all data upon which the assumptions are based are taken directly from SRAFRC studies on the Susquehanna River, or from comparable data on American shad in the Connecticut River.

Although specifically developed for the Susquehanna River, the model could be used on other river systems by appropriate modification of the input data and assumptions.

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APPENDIX A - DEVELOPMENT OF THE MATHEMATICAL MODEL

A.1 - BASIC ASSUMPTIONS

Parent stock = number of adult fish entering Conowingo tailrace + hatchery contribution.

Stock recruitment ratio = 2.0 recruits: 1 parent (Acres, 1991).

Recruits return at ages based on overall average age structure 1980-1989 (Acres, 1991).

II	=	0.23%	V	=	33.77%
III	=	13.02%	VI	=	10.85%
IV	=	40.54%	VII	=	1.60%

A.2 - MAXIMUM SPAWNING AREA CAPACITY

Assuming 205 spawners per hectare (St.Pierre, 1979; Crecco and Savoy, 1987).

Area	Length (km)	Width (m)	Area (hectares)	Maximum Spawners
Conowingo Tail-race & Lower Susquehanna	16	1,220	1,960	400,000
Upper Conowingo Reservoir	4.8	914	442	90,000
Upper Holtwood Reservoir	1.6	1,370	220	45,000
Upper Safe Har-bor Reservoir and River Reach to York Haven	21	1,065	2,230	460,000
York Haven Reservoir & Upper Suquehanna	320	457	14,620	3,000,000

A.3 - HATCHERY CONTRIBUTION

Number of females required in wild to produce hatchery releases = 7,000 (Acres, 1991)

7,000 females x 3 (sex ratio 2 males:1 female) = 21,000 (total hatchery "parents")

A.4 - NUMBER OF "EFFECTIVE SPAWNERS" IN PARENT STOCK

PS = (TR x .95) + (CR) + (CO - HO) + (HO - SA) + (SA - YO) + (YO) + (HA) - Adjustment for habitat

Where: PS = Parent stock
TR = fish transported upstream to York Haven
CR = fish remaining in Conowingo tailrace
CO = fish passage over Conowingo Dam
HO = fish passage over Holtwood Dam
SA = fish passage over Safe Harbor Dam
YO = fish passage over York Haven Dam
HA = hatchery contribution

Assumptions/Limitations:

- CR = Tailrace Population (TP) - number of fish handled by the Conowingo lift
- All fish passing each dam reach the next upstream dam
- Maximum number of fish handled by each fish lift = tailrace population x lift efficiency (from 50-90%)
- Hatchery contribution (HA) = 7,000 females x 3 = 21,000 fish
- Fish not handled by a lift will drop back and spawn in the available habitat in that river segment
- Fish in excess of maximum spawning capacity of each river segment are "lost" to the population

A.5 - TO CALCULATE NUMBER OF RECRUITS

RE = PS X SRR

Where: PS = Parent Stock
SRR = Stock Recruitment Ratio = 2.0

A.6 - TO CALCULATE TOTAL POPULATION

TP = (Age II → Age VII)

Where: Age II → Age VII = Number of returning recruits of Year Classes II, III, IV, V, VI and VII

A.7 - TO CALCULATE TAILRACE POPULATION

Tailrace population = Total Population (TP) - Harvest (FH)

Where: FH = 20 percent of TP in years after the tailrace population exceeds 250,000

THE ROLE OF HYDROELECTRIC PROJECT OWNERS IN THE RESTORATION OF AMERICAN SHAD TO THE SUSQUEHANNA RIVER

By Peter S. Foote and Michael F. Dumont, Acres International Corporation, and
Robert B. Domermuth, Pennsylvania Power and Light Co.[1]

ABSTRACT

The American shad (Alosa sapidissima) historically occurred in large numbers in the Susquehanna River, and supported commercial fisheries in the upper Chesapeake Bay and in the Susquehanna River. During the mid-to-late 1800's, the shad population declined, primarily due to the construction of canal feeder dams. Efforts to rehabilitate the shad population were attempted in the 1800's, but were generally unsuccessful. Four hydroelectric dams were constructed on the lower Susquehanna River between 1904 and 1932. Successful fishways were not constructed at that time, and American shad were excluded from most of the river basin with the closure of Conowingo Dam in 1928.

During the 1950's and 1960's several studies, funded in large part by utilities that own hydroelectric projects on the river, were implemented to assess the feasibility of restoring shad to the river. An active program to restore American shad to the river was initiated in 1970. This program has also been funded in large part by the utilities, and has been jointly conducted with state and Federal resource agencies. Major restoration activities implemented to date include: construction of fish collection, passage and transport facilities at the most downstream dam on the river (Conowingo); construction and operation of a shad hatchery; closure of the sport and commercial fisheries for shad in the upper Chesapeake Bay and lower Susquehanna River; and the completion of several studies related to fish passage at the hydroelectric projects, and other aspects of the restoration program. Results of the restoration program to date indicate that the program has produced an

Peter S. Foote, Head, Environmental Department and Michael F. Dumont, Project Manager, Acres International Corporation, 140 John James Audubon Parkway, Amherst, NY 14228-1180, Telephone: 716-689-3737 and Robert B. Domermuth, Pennsylvania Power and Light Company, 2 North 9th Street, Allentown, PA 18101, Telephone: 215-774-4849

increase in the American shad population in the Susquehanna River and upper Chesapeake Bay.

INTRODUCTION

The American shad (Alosa sapidissima), the largest member of the herring family (Clupeidae) in North America, is an anadromous species that lives most of its life in saltwater, and enters freshwater to spawn. The shad has been an important commercial and sport fish, and historically occurred in most major Atlantic coast rivers from the St. John's River, Florida to the St. Lawrence River in Canada. The Susquehanna River, which has the largest drainage basin of any U.S. Atlantic coast river, once supported a large run of shad. This run declined, however, to a point where only a remnant population existed by the early 1970's. Modern efforts to restore or rebuild the population, funded in large part by hydroelectric project owners on the river, have been underway since the early 1950's. A review of the historical occurrence of shad within the Susquehanna River will be presented, and the restoration efforts implemented to date described.

HISTORICAL PERSPECTIVE

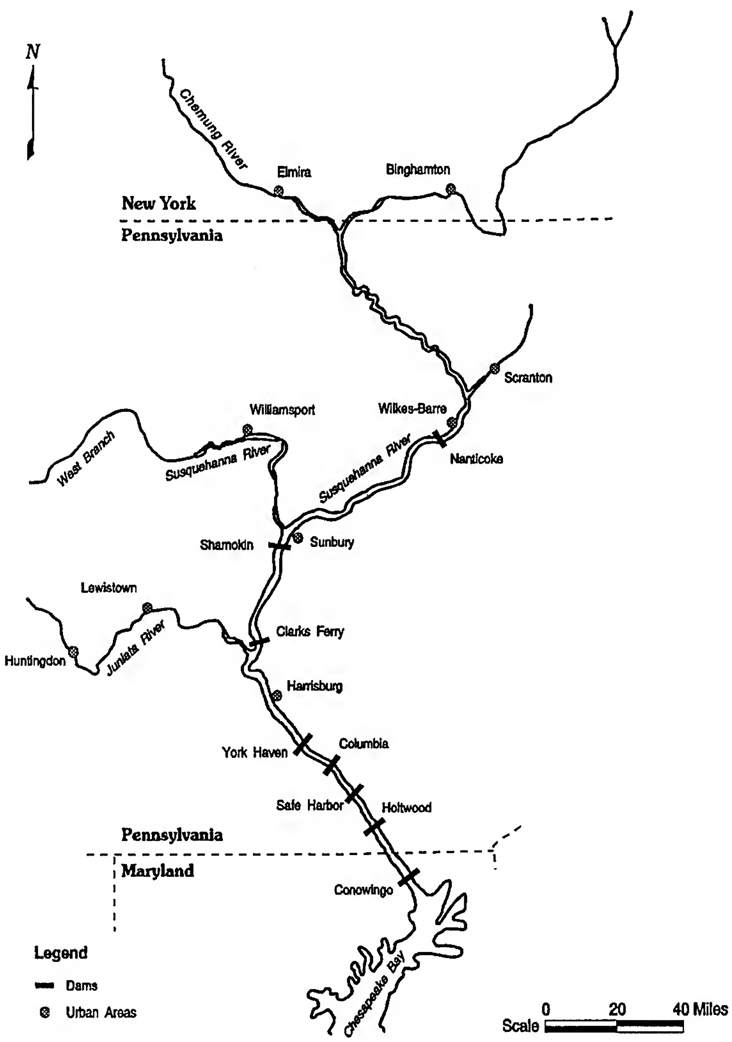
Original Distribution

The American shad once ascended the Susquehanna River to Binghamton, New York, 318 miles from the mouth of the river at the head of Chesapeake Bay (Figure 1). Shad also ascended major tributaries, including the Juniata River. In the 1700's, shad became an important food source for European settlers, and commercial fisheries were established throughout the basin. By the early 1800's, an estimated 2 million pounds of shad (about 670,000 fish) were harvested annually from the Pennsylvania portion of the basin (Walburg and Nichols, 1967). Shad were also landed in the New York waters of the basin, and Susquehanna River shad contributed to shad fisheries in Maryland and Virginia. Gear used in the early Susquehanna fisheries included gill nets, seines, bow nets, and spears. Seines accounted for most of the catch.

Decline of the Fishery

In the early 1800's, construction began on a navigation canal that would parallel the Susquehanna River, for the movement of goods and people between the Chesapeake Bay and Pennsylvania. By the 1830's, several low-head canal feeder dams had been constructed on the Susquehanna River to supply water to the canal. Although most of these dams were only 7 or 8 ft high, they partially or totally blocked upstream mitigation of shad to historic spawning grounds. The canal feeder dam constructed at Columbia, Pennsylvania in 1835, only 43 miles from the mouth of the river (Mansueti and Kolb, 1953), blocked shad migration into most of the basin (Figure 1). Shad runs declined significantly after 1835, and by 1890, only 205,000 pounds (about 68,000 fish) were landed in the Pennsylvania waters of the basin (Walburg and Nichols, 1967).

By the mid-1890's, railroads had replaced the barge canals as the main vehicle of transport. Many of the canal feeder dams fell into disrepair, and were breached by



Major Canal Feeder Dams and Hydroelectric Dams on the Mainstem Susquehanna River Affecting the Movement of American Shad
1830 - 1932

floods or ice, allowing shad to again pass these sites during the spawning runs. Breaching of the Columbia Dam in 1895 allowed shad access to the Susquehanna River 40 miles upstream to Clarks Ferry Dam, and to the Juniata River. A modest revival in the Pennsylvania fishery occurred until the construction of hydroelectric dams in the early 1900's. In 1896, the Pennsylvania fishery harvested 283,000 pounds of shad (about 94,000 fish), and in 1908, 312,000 pounds (about 104,000 fish) were landed.

Hydroelectric project construction began on the mainstem of the river in 1904, with the completion of the York Haven Dam at river mile 65 (Figure 1). This dam ranged in height from 6 to 22 ft, and was believed to be passable by shad at higher river flows (Walburg and Nichols, 1967). In 1910, the 55-ft-high Holtwood Dam was constructed at river mile 25, and restricted shad to the lower 25 miles of the Susquehanna River. Two fishways were constructed at Holtwood Dam, but neither proved successful in passing shad. Commercial fishing continued downstream of Holtwood by seine and dip net until 1921, but catches were relatively small. In 1915, the catch was reported to be 33,000 pounds (about 11,000 fish) (Mansueti and Kolb, 1953).

The 95-ft-high Conowingo Dam was completed in 1928 at river mile 10, preventing shad from reaching the Pennsylvania waters of the basin. A fishway was considered for the project, but the U.S. Commissioner of Fisheries advised the Federal Power Commission that the technology to pass shad over a dam 100 ft high was currently not available (O'Malley, 1923). In lieu of fishways, the States of Pennsylvania and Maryland agreed to receive annual payments from the utility for the "improvement of fishing conditions in the river" (Foote and Robbins, 1977).

The final hydroelectric dam constructed on the lower Susquehanna River was the 55-ft-high Safe Harbor project in 1932. This project, located about 8 miles upstream of the Holtwood Dam, had no effect on the shad migration since fish were prevented from reaching the site by Conowingo and Holtwood Dams. The owners, however, agreed to also make the "in lieu of" payments, since fishways were not installed.

RESTORATION EFFORTS

Early Efforts (1860's - 1930's)

After the construction of canal feeder dams and the elimination of shad from much of the Susquehanna Basin, public sentiment demanded that provisions be made for fish passage at the dams. As a result, the Pennsylvania Legislature passed a law in 1867 that required dam owners on the Susquehanna River to provide fish passage for American shad and other species. This law also established the Pennsylvania Fish Commission, whose mission was to oversee the rehabilitation of the shad run in the Susquehanna River. Between 1867 and 1886, at least four different fishways were constructed at the Columbia Dam (river mile 43) for shad passage. None of the fishways were believed to be successful, and any shad gaining access above the dam likely passed through small breaks in the structure (Mansueti and Kolb, 1953). As noted above, two fishways were constructed at the Holtwood Dam in 1910, but neither successfully passed shad.

Other efforts to rehabilitate the shad fishery included restrictions on fishing gear, especially those that harvested juvenile shad on their outmigration to the sea. Eel weirs, which harvested outmigrating American eels, also collected and killed significant numbers of juvenile shad. These weirs were outlawed in the late 1800's, although enforcement of the law proved difficult for the Pennsylvania Fish Commission (Mansueti and Kolb, 1953).

Artificial propagation of shad, an attempt to supplement wild stocks, began in 1873 with the construction of a shad hatchery on the Juniata River. In 1874, another hatchery was built in Marietta, Pennsylvania, just upstream of the Columbia Dam site. The largest hatchery in the Susquehanna Basin was established in 1877 at Havre de Grace, Maryland, near the mouth of the Susquehanna River. This hatchery operated annually from 1880 to 1916, with annual shad egg collections of up to 210 million. Available information indicates that the shad hatchery operations of this era probably had little effect on maintaining or enhancing shad runs, and most hatchery operations were abandoned in the early 1900's. The Havre de Grace facility ceased operations in 1937 (Mansueti and Kolb, 1953).

1940's

After World War II, interest was renewed within the Pennsylvania Legislature regarding restoration of American shad to the Susquehanna River. In 1949, the Pennsylvania Joint Government Commission conducted a study on installing fishways at Susquehanna River dams. They concluded that the Pennsylvania General Assembly should petition the U.S. Congress to fund an investigation of the biological and hydrological factors involved in the construction of fish passage facilities for shad (Pennsylvania Joint Government Commission, 1949). In 1950, the U.S. 81st Congress appropriated funds for the U.S. Fish and Wildlife Service to conduct a six-year study on the status of Atlantic coast shad fisheries. This appropriation led to a series of studies, some conducted in cooperation with the States of Maryland and Pennsylvania, that laid the basis for current restoration programs.

1950's

Among the several Atlantic coast shad studies conducted in the 1950's was the Walburg 1952 study on the Susquehanna River. Shad were trucked from the head of the Chesapeake Bay in Maryland to stocking locations above the Conowingo and Safe Harbor dams. Walburg's objectives were to determine: if adult shad could be transported above the dams; whether shad would spawn in the river; and whether adult and juvenile shad could survive downstream passage through the hydroelectric projects. Walburg (1954) concluded that adult shad could be successfully transported relatively long distances, but found no evidence of successful spawning by transported adults. Walburg found evidence that a portion of the fish trucked above the hydroelectric dams survived downstream passage through the dams.

From 1958 to 1960, Whitney conducted a second study to determine the desirability and feasibility of passing shad and other species over Conowingo Dam (Whitney, 1961). This effort focused on Conowingo Dam and reservoir, and did

not include upstream portions of the river basin. The study, funded by Susquehanna Electric Company (the owner of Conowingo Dam), was administered by the State of Maryland. Whitney concluded that there would be little benefit in passing shad and several other species over the dam, due to either marginal habitat or the limited potential for increasing the populations of these species. Whitney believed there would be some benefit to the passage of American eel (Anguilla rostrata) elvers, and that a fishway be considered for this species.

1960's

Despite the conclusions of Whitney (1961), two major studies on restoring shad to the Susquehanna River were completed in this decade. The first, an investigation funded by the Pennsylvania Fish Commission, determined the biological and engineering feasibility of installing fish passage facilities on the four hydroelectric dams on the lower river (Bell and Holmes, 1962). Bell and Holmes concluded that fishways would be feasible at the four dams, and could be designed to successfully pass shad and other anadromous species. Several questions remained, however, regarding the feasibility of restoring shad to the river. These included whether the remnant population of shad below Conowingo Dam was sufficient to serve as a "seed stock" for restoration, and whether the Susquehanna River Basin had sufficient suitable habitat to support a significant shad population.

In April 1963, the U.S. Fish and Wildlife Service recommended completion of a study to determine the biological suitability of the Susquehanna River for American shad restoration. In May 1963, the owners of the four lower Susquehanna River hydroelectric dams agreed to fund these studies. Researches conducted the studies between 1963 and 1966, and concluded that most of the Susquehanna River contained suitable shad spawning and rearing habitat (Carlson, 1968). Carlson noted, however, that insufficient information was available to determine if adult shad with the urge to migrate upstream were still available below Conowingo Dam, and whether these fish would successfully utilize fishways and pass through the four lower-river reservoirs.

1970's

The present Susquehanna River American shad restoration program commenced on September 29, 1970, with the signing of an agreement among Philadelphia Electric Power Company (PEPCO), Susquehanna Electric Company (SECO), Pennsylvania Power and Light Company (PP&L), Metropolitan Edison Company (MetEd), Safe Harbor Water Power Corporation (SHWPC), the State of Maryland, the Commonwealth of Pennsylvania, State of New York, and the U.S. Department of the Interior. This agreement provided for:

(1) PEPCO and SECO construction and operation, for a five-year period, of an experimental fish collection and trapping facility in the tailrace of the Conowingo Dam, to determine the number of adult shad in the tailrace that would be available for passage upstream;

(2) PP&L, MetEd, and SHWPC funding of an annual program to collect up to 50 million fertilized shad eggs from the lower Susquehanna and other east-

coast rivers for stocking into suitable waters of the upper Susquehanna River Basin, to develop a shad population with the urge to migrate into the upper Susquehanna River Basin; and

(3) Formation of a Susquehanna Shad Advisory Committee (later renamed the Susquehanna River Anadromous Fish Restoration Committee) to guide implementation of the program.

The experimental Conowingo Dam fish lift was constructed in 1971-72, and first operated in 1972. After the first 5 years of operations, SECO and PEPCO elected to voluntarily continue operations. This facility is still active, and is utilized to collect shad from the Conowingo tailrace, for transfer to upriver release locations.

The shad egg collection program began in 1971. Initially all eggs were stocked into the Susquehanna River or suitable tributaries. In 1976, the Pennsylvania Fish Commission began operation of a utility-funded shad hatchery (Van Dyke Station) on the Juniata River, to determine if hatchery propagation could improve the survival of shad eggs and larvae stocked into the river. This hatchery was subsequently expanded and continues to operate.

By the mid-1970's, the Federal Power Commission (named later changed to the Federal Energy Regulatory Commission - FERC) licenses for the four Susquehanna River projects expired, and the utilities applied for new operating licenses. The States of Maryland and New York, the Commonwealth of Pennsylvania, the U.S. Department of the Interior, the Susquehanna River Basin Commission, and other interested parties intervened in the relicensing process, and requested that FERC order the installation of fishways at the four projects.

Despite this legal challenge, the utilities and agencies continued their cooperative restoration efforts by updating the Bell and Holmes (1962) fishway study, and developing conceptual fishway designs for all four projects (Gilbert Associates, 1978; Harza, 1979).

1980's

The FERC simultaneously issued new operating licenses for the four lower-river projects on August 14, 1980. FERC concluded there was insufficient information to rule on the fish passage issue, and ordered an evidentiary hearing to address this issue. Following 3 years of hearings and negotiations, a settlement agreement was signed on December 7, 1984 among PP&L, MetEd, SHWPC, the state and Federal agencies, and other intervenors. This agreement provided for a 10-year, $3.7 million program, funded by the three utilities, to continue restoration efforts. This allowed further expansion and improvement of the shad hatchery program, expansion of the adult shad trucking program, and studies related to downstream fish passage. The objective of the 10-year program was to determine if self-sustaining populations of anadromous fishes, particularly American shad, could be restored to the river.

PEPCO and SECO were not parties to the 1984 agreement, but continued to maintain and operate the Conowingo Dam fish collection and trucking facility as part of the restoration effort.

In 1980, the Maryland Department of Natural Resources closed the American shad sport and commercial fishery in Maryland waters of the Chesapeake Bay, except the Potomac River. The intent was to allow shad stocks to rebuild through natural production and through restoration efforts, while eliminating the mortality associated with the fishery. The fishery remains closed to the present. Beginning in 1980, the Maryland Department of Natural Resources has also made annual adult shad population estimates for the upper Chesapeake Bay and lower Susquehanna River, and since 1984 has estimated the number of adult shad entering the tailrace of Conowingo Dam.

During the 1980's the Susquehanna River Anadromous Fish Restoration Committee established population restoration goals for the major anadromous fish species that occur in the Susquehanna River. These goals were based on a historical review of the Susquehanna River populations by St. Pierre (1981), and are stated as the number of fish to be passed annually at the Conowingo Dam:

- American shad: 3,000,000; and
- River herring: 20,000,000 (combined).

River herring are a combination of alewife (Alosa pseudoharengus) and blueback herring (Alosa aestivalis).

In 1989, after negotiations with the state and Federal fishery agencies, PEPCO and SECO agreed to construct a permanent fish passage facility at the Conowingo Dam. This facility would have the capability to lift fish directly over the dam (unlike the facility built in 1972), or to continue the trap and trucking program. As a result of this agreement, and pursuant to provisions of the 1984 settlement agreement, the U.S. Fish and Wildlife Service requested that PP&L, MetEd, and SHWPC initiate conceptual design of fish passage facilities at the Holtwood, Safe Harbor, and York Haven Projects.

1990's

The new Conowingo Dam fish lift was completed in 1991 at a cost of about $12 million. During operations in 1991 and 1992, this facility nearly doubled the number of adult shad collected annually from the tailrace (Table 1).

In 1990-91, PP&L, MetEd, and SHWPC funded an investigation of potential fish passage designs at the Holtwood, Safe Harbor, and York Haven projects, and a biological review of shad population data on the Susquehanna River (Acres, 1991). This study included the conceptual design and cost estimates for fish passage facilities at the three projects, based on agency recommendations and on the contractor's assessment of the most appropriate designs. These facilities would not be built simultaneously at all projects, but would be phased depending on the number of fish approaching each dam.

In October 1992, the three utilities and state and Federal fishery agencies agreed on a timetable for construction of fish passage facilities at the Holtwood, Safe Harbor, and York Haven projects. The utilities agreed to construct: two fish lifts at Holtwood by 1997; one fish lift at Safe Harbor by 1997; and one fish lift at York Haven by 2000.

TABLE 1

CONOWINGO DAM FISHLIFT COUNTS, CONOWINGO TAILRACE AND UPPER BAY POPULATION ESTIMATES[1]

Year	Fishlift Counts	Tailrace Estimate	Upper Bay Estimate
1972	182	--	--
1973	65	--	--
1974	121	--	--
1975	87	--	--
1976	82	--	--
1977	165	--	--
1978	54	--	--
1979	50	--	--
1980	139	--	5,531
1981	328	--	9,357
1982	2,039	--	37,551
1983	413	--	12,059
1984	167	3,516	8,074
1985	1,546	7,876	14,283
1986	5,195	18,134	22,902
1987	7,667	21,823	27,352
1988	5,146	28,714	38,386
1989	8,218	43,650	75,820
1990	15,719	59,420	123,830
1991	27,227[2]	83,990	139,862
1992	25,721	86,416	105,255

[1]Sources: SRAFRC (1992a) and (1992b)
[2]Second fishlift placed into operation.

RESULTS OF RESTORATION EFFORTS TO DATE

Conowingo Dam Adult Shad Collections

When the first Conowingo fish facility opened in 1972, only small numbers of American shad ascended the Susquehanna River to the Conowingo tailrace (Table 1). Numbers remained low until the mid to late-1980's, when a significant increase occurred. In 1991, the second Conowingo fishlift went into operation, and the total catch nearly doubled from the previous year. The fishlift counts indicate that the shad population has significantly increased since 1972.

State of Maryland Population Estimates

Both the upper Chesapeake Bay and Conowingo tailrace population estimates indicate a significant shad population increase since the mid-1980's, with the highest populations estimated after 1990 (Table 1).

POTENTIAL FOR SUCCESSFUL RESTORATION

Available information on the size of the shad population in the upper Bay and lower Susquehanna River indicates that restoration efforts over the past 20 years have succeeded in substantially increasing the population of American shad. The utilities have contributed significant funding for these restoration efforts, and have earmarked substantial additional funds for fish passage construction. With this commitment, and assuming that shad returning to the river will successfully spawn in the new habitat being made available by the proposed fish passage facilities, the potential for success appears to be good.

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FISHERIES STUDIES IN THE VICINITY OF THE PROPOSED HARRISBURG HYDROELECTRIC PROJECT

By Lawrence M. Miller[1], Peter S. Foote[2],
Acres International Corporation
Harold M. Brundage, III[3], Environmental Research and Consulting
Thomas R. Payne[4], Thomas R. Payne & Associates and
Daniel R. Lispi[5], City of Harrisburg, PA Department of Public Works

ABSTRACT

The proposed Harrisburg Hydroelectric Project is a 17-foot-high, 34-MW low-head project to be located on the Susquehanna River in Harrisburg, Pennsylvania. A major concern of State and Federal resource agencies reviewing the project proposal was the potential effects of the proposed run-of-river reservoir on fisheries resources in the project area. In response to these concerns, several fisheries studies were conducted to gather data at the proposed project site for use in predicting potential project impacts. Investigations included baseline fisheries studies, smallmouth bass habitat utilization and movement studies, and pre- and post-project fisheries habitat evaluations. The results of these studies and the potential effects of the project are described.

INTRODUCTION

The proposed Harrisburg Hydroelectric Project is located on the Susquehanna River, largely within the corporate limits of the City of Harrisburg, Pennsylvania. The project will require the construction of a 17-ft high gated spillway structure downstream of an existing bridge, and installation of hydroelectric generating facilities (Figure 1). The project will have a total installed capacity of 34.4 megawatts (MW) and will produce an average of 150 gigawatt hours (GWh) of electricity annually. Two powerhouses, each with two 8.6-MW units, will be located on either side of City Island. The development will be operated in a run-of-river mode.

Lawrence M. Miller[1], Environmental Scientist and Peter S. Foote[2], Head, Environmental Department, Acres International Corporation, 140 John James Audubon Parkway, Amherst, NY 14228-1180 (Telephone: 716-689-3737), Harold M. Brundage, III[3], President, Environmental Research and Consulting, 112 Commons Court, Chadds Ford, PA 19317 (Telephone: 215-558-1662), Thomas R. Payne[4], President, Thomas R. Payne & Associates, 850 G Street, Arcata, CA 95521 (Telephone: 707-822-8478), and Daniel R. Lispi[5], Project Manager, City of Harrisburg Department of Public Works, City Government Center, Suite 212, Harrisburg, PA 17101 (Telephone: 717-255-6455)

The proposed impoundment will be approximately 8 miles long and will have a maximum width of about 3/4 mile at a normal operating level of 306.5 ft (mean sea level). The total water surface area at the proposed normal operating level will be 3,800 acres.

Several fisheries studies were conducted by Acres International Corporation (Acres) and its subconsultants, as part of the licensing process required by the Federal Energy Regulatory Commission (FERC). These studies were to be utilized to predict the impacts of the project on fisheries resources, and included: (1) baseline fisheries sampling (1986, 1991, 1992), (2) smallmouth bass habitat suitability and movement studies (1989, 1991, 1992), and (3) pre- and post-project fish habitat evaluation (1988, 1991).

The objectives of this paper are to: (1) summarize the results of these investigations, and (2) discuss potential impacts of the project on fisheries resources.

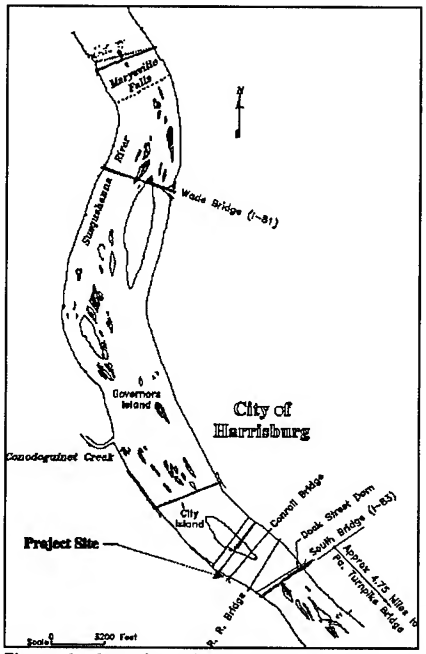

Figure 1. Location of the proposed Harrisburg Hydroelectric Project

DESCRIPTION OF PROJECT AREA

The Susquehanna River through the City of Harrisburg is a wide, shallow and generally slow-moving warmwater river. The existing 5-ft high Dock Street Dam forms a shallow impoundment that is about one mile in length; the remainder of the river within the city limits is interspersed with shallow and deep runs and islands. Small riffle areas occur at Marysville Falls and at cross river ledges between the mouth of the Conodoguinet Creek and Maclay Street (Figure 1).

This reach of the Susquehanna River presently supports a diverse fish community of warmwater/coolwater fish dominated by centrarchids (e.g. sunfish, bass), and cyprinids (e.g. minnows, carp). The area supports a significant sport fishery for smallmouth bass (Micropterus dolomieui), channel catfish, (Ictalurus punctatus), and several species of sunfishes. Walleye (Stizostedion vitreum) and muskellunge (Esox masquinongy) are sometimes taken by sport fishermen in the area. The primary species of interest in the Harrisburg Project area is the recreationally important smallmouth bass.

Fisheries investigations occurred throughout the 8-mile-long proposed project reach, and within the existing York Haven Project impoundment, located about 10 miles downstream of Harrisburg on the mainstem of the Susquehanna River. This impoundment is about 3.7 miles long and is similar in depth, and in the availability of upstream riverine habitat, to the proposed Harrisburg Project impoundment. The York Haven reach may, therefore, be analogous to post-project conditions at Harrisburg, and served as a post-project "model" for comparison and impact prediction.

BASELINE FISHERIES STUDIES

Methods

In 1986, 1991 and 1992, Acres and its subconsultant conducted a fisheries sampling program in the proposed project area, to provide baseline fisheries information. Electrofishing, trap netting, gill netting, and seining techniques were employed. Sampling during the 1986 study was restricted to the Harrisburg project reach. In 1991 and 1992, fisheries sampling was conducted in both the Harrisburg project reach and in the York Haven Pool, to gather data for comparison of the two study areas. Electrofishing results were the primary data utilized for comparison.

Sampling locations were partitioned according to the dominant habitat types (i.e. habitat zone) occurring at that location, so that relative abundance of fish in a particular habitat type could be assessed. Annual, seasonal and day/night relative abundance of fish (reported as catch per minute of electrofishing) were also tabulated. General population structure for smallmouth bass was analyzed using length-frequency distributions, and length at age data were determined for smallmouth bass by fish scale analysis in 1992.

Results

A total of 46 species of fish was collected in the Harrisburg project area. With the exception of the cyprinid species, smallmouth bass was numerically the most abundant species found during all sample years, seasons and habitat zones in both the Harrisburg and York Haven areas (Table 1). The next most abundant species were other members of the centrarchid family. Gamefish and other recreationally important species that contributed to more than one percent of the total catch included smallmouth bass, redbreast sunfish (Lepomis auritus), and rock bass (Ambloplites rupestris). Other game or recreationally important species collected were channel catfish, American shad (Alosa sapidissima), Amur River pike (Esox reicherti), muskellunge, largemouth bass (Micropterus salmoides), walleye, black crappie (Pomoxis nigromaculatus), white crappie (P. annularis), green sunfish (Lepomis cyanellus), pumpkinseed (L. gibbosus), and bluegill (L. macrochirus).

Table 1: **Electrofishing catch rates (fish/minute of electrofishing) for the most commonly collected species for the Harrisburg project baseline fisheries investigations in 1986, 1991 and 1992.**

SPECIES	1986	1991		1992	
	Harrisburg	Harrisburg	York Haven	Harrisburg	York Haven
Smallmouth bass	0.87	1.67	1.52	2.09	1.27
Rock bass	0.37	0.36	0.36	0.20	0.47
Redbreast sunfish	0.48	0.27	0.48	0.20	0.43
Bluegill	0.03	-	0.39	0.02	0.25
Channel catfish	0.02	0.01	-	0.01	-
Green sunfish	0.01	-	0.03	0.01	0.04
Black crappie	0.01	-	-	<0.01	-
Pumpkinseed	0.01	-	0.27	<0.01	0.25
Muskellunge	0.01	<0.01	-	<0.01	-
Largemouth bass	<0.01	<0.01	0.12	<0.01	0.04
Amur River pike	-	0.01	-	<0.01	-
Walleye	0.04	<0.01	0.10	0.02	0.02
Others	0.22	0.64	1.13	0.61	0.66
TOTAL	2.07	2.97	4.40	3.17	3.43

Seasonal differences in catch rates occurred, with rates typically highest during the spring and fall seasons, and lowest during summer. As with most other species, the night sampling catch rates for smallmouth bass were greater than day sampling catch rates in most sampling cells. In 1986, no electrofishing was performed at night, and this may account, in part, for the lower smallmouth bass catch rates for that year.

Catch rates for all species combined were consistently high in the deep pool habitat zones in both years and in both the Harrisburg and York Haven areas (Table 2). The shallow run/riffle zone also exhibited high catch rates, generally followed by the deep run zone, although York Haven data indicated good catch rates in deep run habitat. Island cluster habitat had the lowest catch rate in 1992, and the second lowest catch rate in 1991, for the Harrisburg study area. Island cluster habitat was not sampled in the York Haven area, since this habitat type was not present.

The overall catch rates for all species combined were greater in the York Haven study area, when compared with the Harrisburg study area, in both 1991 and 1992 (Table 1). In both 1991 and 1992, electrofishing catch rates for smallmouth bass were higher in the Harrisburg study area than in the York Haven study area.

Smallmouth bass population structure for the Harrisburg and York Haven areas were similar. The percent age composition for the two study areas, based on length-frequency data and scale analysis, were as follows:

AREA	AGE									
	1	2	3	4	5	6	7	8	9	10
Harrisburg	62	4	4	10	8	5	4	2	2	<1
York Haven	53	12	10	15	5	3	1	1	0	0

A comparison of the mean length at age data from the Harrisburg and York Haven study areas indicates that growth rates are similar between the two areas (Table 3). When the smallmouth bass length at age results of this study are compared to back-calculated mean lengths derived from other Susquehanna River studies, and to the Pennsylvania state average for lakes (Table 3), it is apparent that bass from this study are growing as well as or faster than those from the comparison studies.

SMALLMOUTH BASS HABITAT SUITABILITY AND MOVEMENT STUDIES

Methods

In 1989 and 1991, SCUBA diving studies were conducted to collect data on smallmouth bass habitat usage, for use in development of habitat criteria curves for smallmouth bass. These curves were to be used in the pre- and post-project habitat evaluation using the Instream Flow Incremental Methodology (IFIM), described below. A goal of fifty habitat observations per life stage (i.e. spawning, juvenile, and yearling/adult rearing) was established for the verification of previously published habitat suitability criteria (Edwards et al. 1983). The study was designed to minimize the biasing effect of limited habitat availability, observer subjectivity, and fish disturbance through the use of equal area sampling within habitat strata, transect randomization, and direct diver observation. Enough data were collected to allow development of site-specific curves for use in the IFIM analysis.

The Harrisburg study area was delineated into shallow pool, deep run, shallow run, riffle, and island cluster habitat units by reference to aerial photographs and existing depth and velocity profiles under low flow conditions. Due to the lack of deep pool habitat in the Harrisburg area, deep pool habitat was delineated in the York Haven study area. Sampling was conducted equally within all habitat types.

Diving transects were selected randomly from within each habitat type, and were 500 feet long and orientated perpendicular to the direction of flow. After a transect was delineated, a diver swam slowly along the transect line, scanning the water column for smallmouth bass. When a bass was observed, the diver recorded its estimated length, focal height above the bottom, focal nose velocity, bottom substrate and nearby instream cover type. Total depth and mean water column velocity were then measured at the point of observation. Transect length and direction, weather conditions, river stage, time of transect dive, water temperature and clarity were also recorded.

Additional data on habitat usage and movement patterns of smallmouth bass in the Harrisburg project area, and in the York Haven study area were collected by a radio

Table 2: Electrofishing catch rates (fish/minute of electrofishing) for all species combined by habitat type for the Harrisburg and York Haven study areas, 1991 and 1992.

	Deep Pool		Island Cluster		Deep Run		Shallow Run/Riffle		TOTAL	
	Harrisburg	York Haven	Harrisburg	York Haven	Harrisburg	York Haven	Harrisburg	York Haven	Harrisburg	York Haven
1991	4.18	3.93	2.22	n/a	1.74	4.80	2.46	4.53	2.97	4.40
1992	4.13	3.39	1.26	n/a	3.31	3.72	5.45	2.85	3.17	3.43

Table 3: Mean length (mm) by age at capture for smallmouth bass collected during April, 1992 electrofishing, and back-calculated mean lengths at age for smallmouth bass captured in other Susquehanna River studies and the average for Pennsylvania lakes.

Study	Age									
	1	2	3	4	5	6	7	8	9	10
1992 Study - Harrisburg	121	207	266	298	328	362	380	400	432	469
1992 Study - York Haven	131	192	251	296	337	350	396	-	-	-
Susquehanna River Section 03 - 1984[1]	99	159	204	254	300	352	407	-	-	-
Susquehanna River Section 04 - 1984[2]	114	178	223	255	-	-	-	-	-	-
Susquehanna River Section 02 - 1984[3]	100	170	218	263	287	324	360	-	-	-
State Average Lakes - 1982[4]	103	173	229	277	326	369	402	-	-	-

[1] Jackson el al (1986a); [2] Jackson el al (1986b); [3] Jackson el al (1986c); [4] Cooper (1982)

telemetry investigation from April through August 1992. Smallmouth bass were tagged with surgically implanted radio transmitters, having a calculated battery life of 90 days. Each tag operated at a discrete frequency, allowing identification of individual fish. Radio-tagged fish were tracked with a programmable scanning receiver. A whip antenna mounted to the gunwale of an aluminum boat was used to determine general locations. More precise locations were determined using a hand-held bi-directional tuned-loop antenna.

A total of 30 smallmouth bass was radio-tagged, 15 in the Harrisburg study area and 15 in the York Haven area. Fish tagged in the Harrisburg reach ranged in length from 310-440 mm (FL) and weighed 0.5-1.60 kg; six were male, eight were female, and the sex of one fish was not determined. Smallmouth bass tagged in York Haven Pool ranged in length from 298-396 mm (FL) and weighed 0.7-1.5 kg; five were male and ten were female.

Water depth and mean water column velocity were measured at each "fix" (the location of the fish as determined by manual tracking with the loop antenna). Dominant substrate type was estimated according to a modified Wentworth scale (0-sand to 8-bedrock) (Bovee and Milhouse, 1978). Macrohabitat at the location was classified as riffle, shallow run, deep run, shallow pool, deep pool, or island cluster. Location was further classified as near shoreline (within 6 m of shore) or off shoreline. The occurrence of velocity shelters (e.g., logs, boulders, beds of emergent vegetation) was also noted. Water temperature, dissolved oxygen concentration, and Secchi depth were measured periodically during each tracking day. River stage and discharge were obtained from the National Weather Service River Forecast Center.

Results

Sufficient habitat suitability data were collected to allow development of site-specific criteria curves for Susquehanna River smallmouth bass. The new curves are believed to be more representative of habitat conditions for Susquehanna River smallmouth bass, and were used in the IFIM habitat assessment.

The radio telemetry study provided additional information on smallmouth bass habitat usage, as well as data on longer-term movement patterns of bass. Fish were grouped into four categories that generally characterize their movement patterns (Table 4). Fish from Categories 1, 2, and 3 appeared to establish home ranges, as several observations occurred in a localized area over an extended period of time. Fish from Category 4, however, did not exhibit this characteristic and may be classified as nomadic (Einhouse 1981).

Table 4: Summary of smallmouth bass movements during the radio telemetry study.

Category	Description	N	%	Comments
1	Localized movement restricted to the area of capture and tagging	23	76	12 fish from Harrisburg area and 11 fish from York Haven exhibited this pattern
2	Long-range movement from the area of capture and tagging to a different location with subsequent localized movement restricted to the new area	3	10	Two fish moved downstream from the Harrisburg area and one fish upstream from the York Haven area to a location near the Harrisburg Airport
3	Long-range movement from the area of capture and tagging to a different location, with subsequent return to the area of capture and tagging where localized movement was then prevalent	2	7	Both fish in this category were from the York Haven area
4	Long-range wandering that did not exhibit a tendency for restricted localized movement	2	7	One fish was from the Harrisburg area and one was from the York Haven area

The majority (76%) of smallmouth bass observed in this study exhibited movement behavior consistent with Category 1. In general, the sample smallmouth bass population did not exhibit a tendency for long-ranging movements. This observation is consistent with other studies conducted on smallmouth bass movement utilizing a variety of monitoring techniques (Reynolds 1965, Munther 1970, Todd and Rabeni 1989). This study did find that there is a small segment of the smallmouth bass population that does exhibit long range movements. Other researchers have also observed this for smallmouth bass and other non-migratory species (Einhouse 1981).

PRE- AND POST-PROJECT FISH HABITAT EVALUATION

Methods

The Instream Flow Incremental Methodology (IFIM) was used to generate indices of habitat (weighted usable area - WUA) for pre-project (existing) and post-project conditions in the Harrisburg study area. Baseline (pre-project) hydraulic data (bottom profile, water velocities, water surface elevations, and substrate composition) were recorded on twenty transects throughout the project area. Pre-project conditions were simulated using the IFG-4 model, which uses one set of measured velocities and water surface elevations, and rating data to simulate velocities and depths for other river flows. Post-project conditions were simulated by imposing post-project water surface elevations (as determined by a HEC-2 analysis) on the same transects, and using the "no-velocity" option of IFG-4. This option distributes velocities across transects based on discharge and individual cell depth.

Potential changes to aquatic habitat after construction of the proposed dam were evaluated by using the Physical Habitat Simulation (PHABSIM) programs, which

utilized both literature-based and site-specific species criteria curves, to produce estimates of WUA. The simulation at each transect is assumed to be representative of available habitat in the vicinity of the transects, and each transect is weighted for the amount of habitat it represents in the total study reach. The transects can then be used to estimate WUA values for the study reach as a whole. The reach of the river evaluated included the eight miles of river upstream of the proposed dam, and the less than one-mile reach between the proposed dam and the existing Dock Street dam. This "below-dam" reach will experience a reduction in water surface elevation as the existing dam is mostly removed as part of the proposed project construction.

Results

Habitat characteristics in the below-dam reach will change from the existing pool conditions to more riverine habitat after the existing Dock Street dam is removed. Characteristics above the new dam will change from the existing riverine habitat to more reservoir-like conditions.

The effects of these habitat changes on WUA for the four indicator species are summarized in Table 5. The WUA analysis indicates that habitat changes would have varying effects on the several evaluation species, depending on whether the location is above or below the proposed dam. In general, for the below-dam reach, habitat value would decrease for more of the species and life stages, although there would be a gain in habitat value for some life stages. For the above-dam reach (the reservoir), there would be a general increase in habitat value for three of the evaluation species, although there would be a decrease in habitat value for a few life stages.

Table 5: Trends in WUA from pre-project to post-project conditions for four evaluation species, above and below the proposed hydroelectric dam, at flows less than 120,000 cfs.

Species	Channel catfish			Smallmouth Bass			American shad					Gizzard shad		
Life stages*	S	J	A	S	J	A	S	E	J	O	I	S	J	A
Below dam	-	+	-	-	+	-	+	+	-	-	-	-	-	-
Above dam	-	-	+	+	-	+	+	-	+	+	+	+	+	+

* S-spawning, J-juvenile, A-adult, E-egg incubation, O-outmigration, I-inmigration
\+ Increase in WUA
\- Decrease in WUA

CONCLUSIONS

Seven years of fisheries investigations in the Harrisburg reach of the Susquehanna River indicate that healthy, dynamic fisheries resources exist in the project area. The dominant fish species in the area, exclusive of cyprinid species, is the smallmouth bass, and this species supports an excellent sport fishery in the area.

If the York Haven impoundment is used as a model for the Harrisburg reach after construction of the project, probable post-project species composition in the Harrisburg area will incorporate more lacustrine characteristics similar to York Haven. Thus, the Harrisburg area will likely still support a good smallmouth bass fishery, while fisheries for other centrarchid species and walleye should improve.

Information from the smallmouth bass telemetry study indicates that the majority of adult sized smallmouth bass do not make long-distance movements, and tend to restrict their movements to localized areas. Smallmouth bass in York Haven generally remained within the impoundment, with the exception of a few fish that exhibited longer-range primarily upstream movements. These studies indicate that adult smallmouth bass should readily adapt to post-project shallow pool habitat, while the potential for turbine entrainment of adult bass moving downstream through the project would be low.

Analysis of pre- and post-project fisheries habitat conditions indicates that decreases in habitat for some life stages above or below the dam could be offset by increases in habitat for other life stages. In general, species that prefer reservoir-like habitat may be somewhat enhanced by the proposed project reservoir, although species preferring riverine habitat will continue to occur in the project area due to the relatively close proximity of upstream riverine habitat.

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Long Term Hydroacoustic Evaluations of a Fixed In-Turbine Fish Diversion Screen at Rocky Reach Dam on the Columbia River, Washington

Tracey W. Steig[1]
Bruce H. Ransom[1]

Abstract

Hydroacoustics were used to monitor downstream migrating juvenile salmon and steelhead trout at Rocky Reach Dam on the Columbia River. Results were used to evaluate and optimize a prototype bar screen deflector's ability to divert migrants from a turbine intake. Specific objectives were to estimate vertical distributions, velocity distributions, and approach trajectories of fish as they approached and entered Turbine Unit 1.

Single-beam and dual-beam hydroacoustic techniques were employed, utilizing manual echogram digitizing and automatic echo tracking techniques. Up to 14 hydroacoustic transducers were deployed to characterize in-turbine fish movement and behavior. Data were collected 24 h/d during the 1989-1992 spring outmigrations, from mid-April through May each year.

Several bar screen configurations were tested, consisting of different screen designs, porosity, and bar screen materials. There was also a baseline configuration in which the bar screen deflector was not installed in the turbine intake. Hydroacoustic data were analyzed by individual configuration. The resulting distributions were compared visually and statistically to each other to determine if changes in test configurations caused any changes in fish distributions or behavior which might alter fish guidance efficiency (FGE).

The densities of fish above the bar screen deflector following FGE tests were calculated. The results showed high fish densities immediately following the FGE test. Densities of fish decreased dramatically thereafter. This suggested that there were large numbers of fish that were guided by the screen but not collected by the mechanical fish extractor (i.e., dip basket).

1 Hydroacoustic Technology, Inc., 715 NE Northlake Way, Seattle, WA 98105-6429; (206) 633-3383

This was because the fish were in the area above the bar screen deflector (i.e., guided), however no longer in the trashrack slot and not accessible by the dip basket. The extractor-based estimate of FGE would more than double if these fish were included in the estimate of guided fish.

Two transducers were used to estimate the percent of fish moving toward and away from the trashrack slot during turbine operation. The results showed that 13-37% of the detected fish were traveling out of the trashrack slot. During the configurations when the bar screen deflector was in the fishing position, 21-37% of the fish were traveling away from the trashrack slot. During configurations when the bar screen deflector was either removed or in the neutral position, 13-26% of the fish were traveling away from the trashrack slot. The percentage of detected fish that were guided ranged from 26% to 75% for the various test configurations.

The hydroacoustic estimate of FGE is dependent on the numbers of fish passing above the bar screen deflector. Since there were indications that fish were detected multiple times above the screen, it was important to quantify the number of fish exiting the trashrack gatewell. The percentage of fish exiting the trashrack gatewell was estimated by the paired single-beam data analysis that determined the directionality of the fish. This was done in an attempt to avoid overestimating FGE.

The potential hydroacoustic estimates of FGE (adjusted for only guided fish) were calculated for 24-hr periods, and ranged from 29% to 56%.

Introduction

Since the early 1950's, the salmon and steelhead trout runs (*Oncorhynchus spp.*) on the Columbia and Snake rivers in Washington State have declined due to several factors, including the construction and operation of hydroelectric dams. In the last decade, considerable effort has been devoted to restoring and enhancing these fish runs. One such method investigated by Chelan County Public Utility District No. 1 is the use of in-turbine diversion screens to guide and bypass downstream migrants. In-turbine screens have been found to be a relatively safe means of diverting, collecting and passing migrants at other hydroelectric dams. Knowledge of the fish distributions and behavior are essential for making an informed evaluation of the effectiveness of the in-turbine screens as a bypass method.

The primary objective of the studies reported here was to evaluate the in-turbine screen's ability to diver the downstream migrants out of the turbine intake. During the 1989-1991 outmigrations, a prototype bar screen deflector was installed and evaluated in the upstream (trashrack) gatewell of the turbine intake. In 1992, a prototype bar screen deflector was installed and evaluated in the downstream (headgate) gatewell.

Site Description

Rocky Reach Dam is located on the Columbia River 7 miles north of Wenatchee, Washington at river mile 475. The dam's spillway is perpendicular and its powerhouse parallel to river flow (Figure 1). The powerhouse is 1088 ft long and contains 11 vertical Kaplan turbines

numbered from south to north. Turbine Units 1-7 have a rated capacity of 116 MW each. Turbine Units 8-11 are each rated at 125 MW. Each turbine has three rectangular intakes, 20 ft wide by 50 ft high at the headgate gatewell. The spillway is over 750 ft long and has 12 automatic spill gates. Each gate is 50 ft wide and approximately 60 ft deep to the spill gate ogee.

All hydroacoustic evaluations were performed at Turbine Unit 1, the southermost unit of the powerhouse. Between 1989-1992, three prototype bar screen deflectors were installed and tested in the north and south intake slots of Turbine Unit 1 (Figure 2).

Methods

Over the last two decades, hydroacoustic technology has been developed to allow accurate measurements of fish abundance, distribution, directionality, size, and behavior (Ransom 1991).

Three different types of hydroacoustic systems have been used at Rocky Reach Dam. The single-beam system consisted of the following components: 420 kHz transducers, two echo sounder/transceivers, two multiplexer/equalizers, three chart recorders, and an oscilloscope. The dual-beam system consisted of the following components: 200 kHz transducer, an echo sounder/transceiver, a digital signal processor, a digital chart recorder, and an oscilloscope.

Calibration and operation of the hydroacoustic systems are described in detail by Albers (1965) and Urick (1975).

Data were collected 24 h/d from mid-April to the end of May, from 1989-1992.

Individual transducers were placed at various underwater locations immediately upsteam of Turbine Unit 1 (Figures 3 and 4). Circular and elliptical transducers were used and had nominal beam widths of 2°, 6°, 4° x 12°, and 6° x 15°.

Single-beam data were displayed on chart recorder echograms. Echogram traces had to satisfy three criteria to be classified as downstream migrants: 1) the strength of target echoes had to exceed a predetermined threshold; 2) the targets had to be detected by consecutive pulses (i.e., exhibit redundancy); and 3) targets had to show movement toward the intake.

Microcomputers were used for data storage and analysis. Data from individual fish detections recorded on the echograms were transformed to numeric data files on a microcomputer by using a digitizing pad and appropriate software. Computer programs were developed to analyze vertical distributions, trajectory angle distributions and velocity profile distributions.

The dual-beam hydroacoustic system and processor digitized the analog signals and automatically processed the data. These echo signals were relayed to the HTI *Model 300 Digital Echo Processor* (*DEP*), a computer-based dual-beam processor that detects individual fish. The

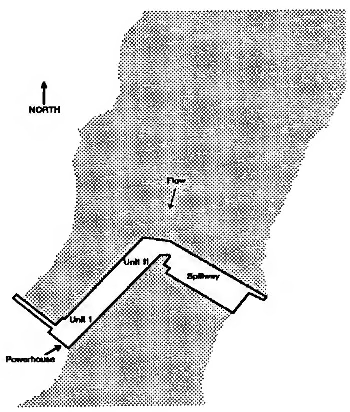

Figure 1. Plan view showing the orientation of Rocky Reach Dam on the Columbia River.

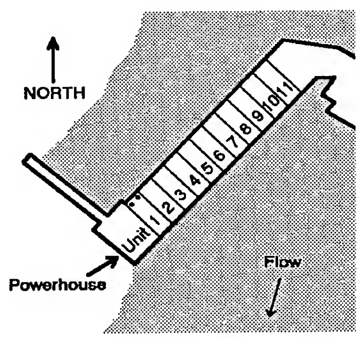

Figure 2. Plan view of Rocky Reach Dam powerhouse showing transducer mounting locations used during the springs of 1989-1992.

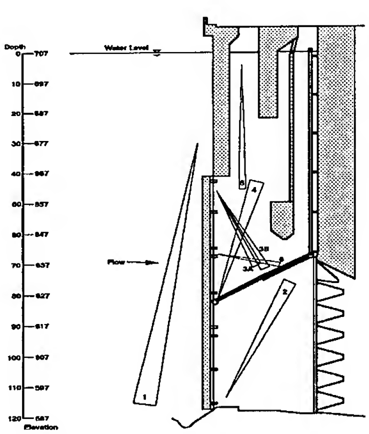

Figure 3. Cross-section of Rocky Reach Dam at Unit 1, south intake showing the various transducer mounting locations.

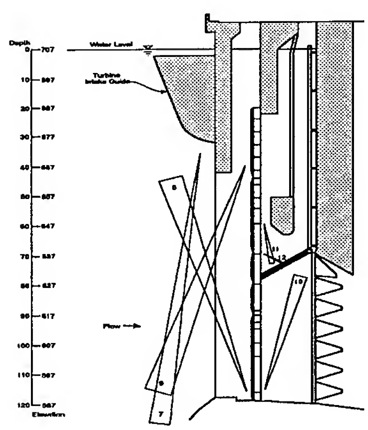

Figure 4. Cross-section of Rocky Reach Dam at Unit 1, north intake showing the various transducer mounting locations.

detections are then analyzed to estimate vertical distributions, and fish acoustic size (i.e., target strength) for each fish monitored.

The effectiveness of any fish diversion technique is dependent on the position of the fish in the water column, and their behavioral response to the bypass device. The vertical distributions, velocity profiles, and size distributions of the downstream migrants provide information about their behavior as they approach and pass through turbine intakes.

To characterize the distribution and trajectory of the migrants as they approached the bar screen deflector, three types of primary results were calculated for each of the single-beam transducers monitored. These included vertical distributions, average trajectory angles, and average target velocities.

The vertical distributions were calculated from the ranges at which a fish entered and exited the acoustic beam. Each distribution, as a function of range from the transducer, was estimated as the percentage of fish passing through the acoustic beam for each 1-ft stratum. The percentages for each stratum were then combined to generate a cumulative distribution.

Each fish's entrance and exit ranges and time in the beam are written to a data file. The midpoint range is used to categorize each fish into the appropriate range strata. The average entrance and exit ranges for each range strata and the average time in the beam were calculated. The average trajectory angle of the fish passing through the beam for each range strata was calculated from the transducer beam width and the average entrance and exit ranges. The average chord length (fish distance traveled) of this elliptical plane was then calculated.

The fish velocity was calculated by dividing the average chord length by the average residence time in the beam (for each range strata). For each configuration, velocity and trajectory profiles were graphically plotted for each transducer sampled.

Another data analysis method was developed to estimate the relative density of fish within a semi-confined area with limited water flow, such as within gatewells. The density of fish was proportional to the amount of time all of the monitored fish remained within the acoustic beam (weighted for beam spreading) per unit of sample time. The results provided indications of changes of fish densities over time. A fish density function was appropriate for this application since the fish in the trashrack gatewell were exhibiting a milling behavior.

Another hydroacoustic technique was developed to better assess fish directionality through the acoustic beam. This was accomplished by physically mounting two single-beam transducers at slightly different angles (difference in aiming angles 2°-3°), and fast multiplexing between them. The fish targets were then tracked through both acoustic beams. Knowing which beam the fish entered and exited first provides the directionality of the fish. This technique has been widely used with dual-beam transducers and digital signal processors (Johnston and Hopelain 1990), and was successfully used

with a single-beam system at Rocky Reach Dam in 1991 (Steig 1991) and Rock Island Dam in 1992 (Steig 1992).

Further data analysis estimated the average slope, the average range, and the percentage of fish traveling in each direction. This information was then used in conjunction with other results.

Results and Discussion

The hydroacoustic studies conducted at Rocky Reach Dam from 1989-1992 included a wide variety of objectives and an equally wide range of results. This paper includes some examples of typical results from these studies. The project reports from these studies include more in-depth analysis and results (Steig 1990, 1992, 1993; Steig and Ransom 1989).

Figure 5 shows an example of fish moving through the two acoustic beams toward and away from the trashrack slot. Figure 6 shows an actual example of an echogram with the traces of the fish as they passed through the two acoustic beams. The results of one study showed that between 13% to 37% of the detected fish were traveling out of the trashrack slot. The percentage of detected fish that were guided ranged from 26% to 75%, depending on the various test conditions.

Taking the above information into account, the potential FGE estimated by hydroacoustics ranged from 29% to 56%. This was much greater than was estimated by net catch data. This suggested that if more of the guided fish could be collected, the actual FGE would be significantly higher.

Following FGE tests in 1991, the transducers above the bar screen were monitored for approximately 1 hr after the unit was shut down. The objective was to determine if there were a large number of fish above the bar screen deflector (i.e., guided fish) immediately following the FGE test. The fish densities were calculated from data collected at the transducers monitoring above the bar screen deflector.

The example presented here is from one FGE test. The turbine unit was shut down at 1402 h and continuous monitoring was started at 1404 h. Fish densities were high immediately following the FGE test (Figure 7). The densities of fish decreased dramatically over the next 15 minutes, and normalized during the last 25 minutes.

The high density of fish above the bar screen deflector following the FGE test suggests that there were large numbers of fish that were guided but not collected. In this example, the net-derived FGE estimate was 13%. However, taking into account the acoustically observed fish swimming above the screen following the FGE test, the estimated FGE would have been 28% if those fish could have been collected.

The tracked fish data from dual-beam monitoring were processed to calculate the target strength frequency and fish passage distributions. The target strength and fish passage as a function of range are presented in Figure 8. The results showed that fish passage rates decreased with range

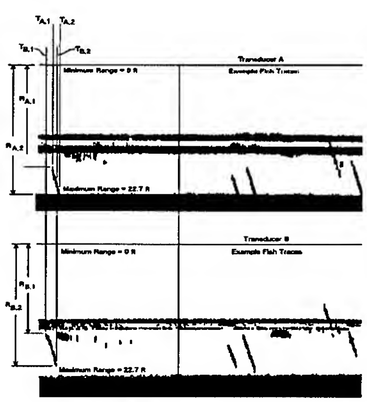

Figure 5. Example of echogram of the paired single-beam transducers and the tracking method used. The traces shown on the left of the echogram show that the fish entered Transducer B ($T_{B,1}$ is before $T_{A,1}$) and entered the Transducer B at a shorter range than Transducer A ($R_{B,1}$ is shorter than $R_{A,1}$).

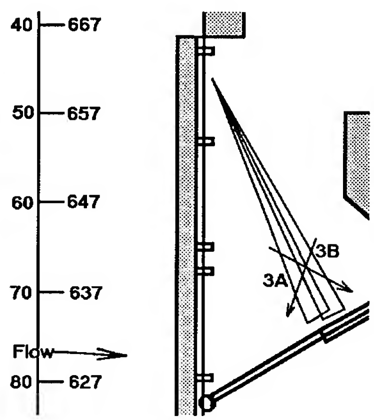

Figure 6. Example of the average range and the estimated trajectory moving through the two acoustic beams, toward and away from the trashrack slot.

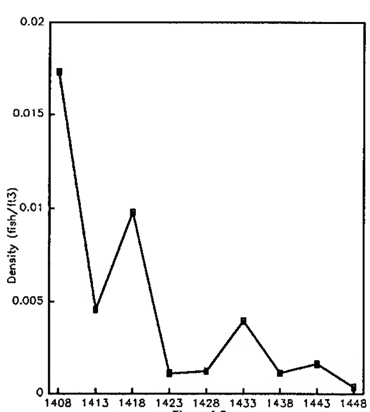

Figure 7. The density of fish above the bar screen deflector following an FGE test.

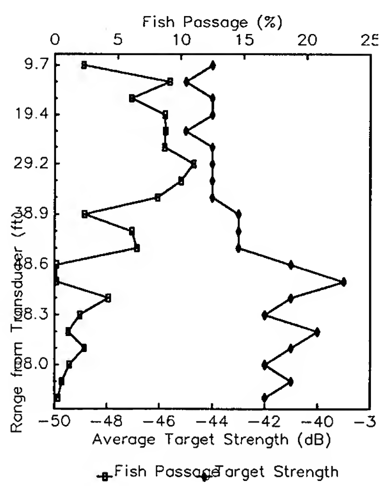

Figure 8. Target strength and fish passage as a function of range from the transducer.

from the transducer. The results also showed that target strength increased slightly with range from the transducer (-44 dB to -40 dB). This suggests that slightly larger fish passed deeper in the water column.

Data were collected at two complimentary transducers with similar aiming angles and mounting locations in front of the north and south slots of Turbine Unit 1 (Figure 12). The data were expected to provide a good comparison between the distributions and trajectory of fish entering two different intakes. The velocity , trajectory, and vertical comparisons between the two intakes are presented in Figures 9, 10, and 11, respectively. For the velocity distribution, trajectory angle profile, and vertical distribution, there were statistically significant differences between the fish approaching the north intake slot with the fish approaching the south intake slot of Unit 1. The fish approached the south intake slot with a higher velocity, steeper trajectory and deeper in the water column than the north intake slot.

Conclusions

Hydroacoustic techniques were used to evaluate and optimize a bar screen deflector's ability to divert migrants from a turbine intake. One method was to estimate the densities of fish above the bar screen deflector following FGE tests. The results showed high fish densities immediately following the FGE tests. Densities of fish decreased dramatically thereafter. This suggested that there were large numbers of fish that were guided by the screen but not collected by the mechanical fish extractor (i.e., dip basket). The fish in the area above the bar screen deflector were guided, but were not accessible to the dip basket. The estimate of FGE would have more than double if the guided fish that were not extracted were included in the estimate of guided fish.

Another method estimated fish directionality. This technique showed that there were fish traveling out of the trashrack slot. This suggested the bar screen deflector performed better than was reflected in the results of the collected (guided) fish.

Finally, utilyzing the previous results, hydroacoustic estimates of FGE were made. These hydroacoustic estimates of FGE ranged from 29% to 56%, and were greater than the net-derived estimates of FGE.

Acknowledgements

This study was funded by Chelan County Public Utility District No. 1. The authors would like to thank the following Chelan County PUD personnel for their able assistance and advice throughout this study: Richard Nason, Steve Hays, Chuck Peven, Keith Truscott, Barry Keesee, Lance Lorrain and the diver mechanics crew from Central Maintenance. We would also like to thank Dr. John Skalski of the Center for Quantitative Science at the University of Washington for his statistical analysis of the distributional data.

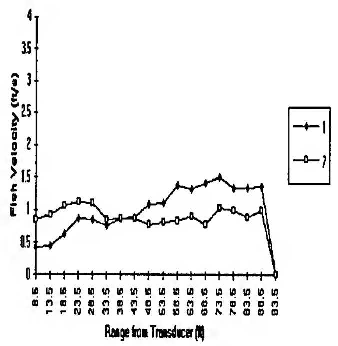

Figure 9. Fish velocity comparisons between the south intake slot (1) and the north intake slot (7) of Unit 1.

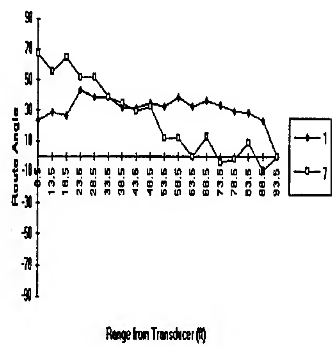

Figure 10. Trajectory angle comparisons between the south intake slot (1) and the north intake slot (7) of Unit 1.

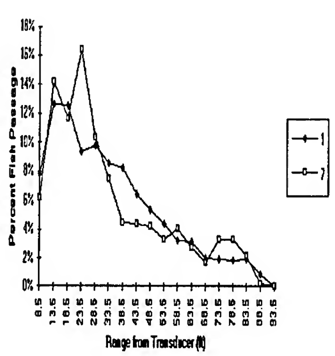

Figure 11. Vertical distribution comparisons between the south intake slot (1) and the north intake slot (7) of Unit 1.

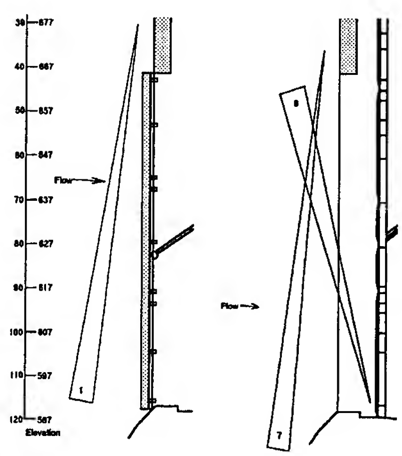

Figure 12. Comparisons of the cross-sections of the south and north intake slots of Unit 1.

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CASE STUDY - 401 PERMIT AND DISCHARGE AERATION

Bryan R. Maurer[1], Daniel J. Barton[2],
and Paul C. Rizzo[3]

ABSTRACT

During the summer and early fall, water entering the Youghiogheny Hydroelectric Project in southwestern Pennsylvania typically falls below the 401 Permit minimum dissolved oxygen requirement of 7.0 mg/l due to thermal stratification of the reservoir. As a result, detailed investigations of discharge aeration systems were implemented, including the evaluation of surface and submerged aerators, wheel gate bypass, turbine venting systems, and draft tube air injection systems.

Ultimately, low pressure air injection was deemed most feasible for the project. Final implementation of this system involved injection of air to the project draft tubes with an injection capacity of about 2000 cfm of air. This system allows for compliance with the water quality requirements of the FERC License while maintaining power generation during the summertime lake stratification. The system has been demonstrated to add up to 5.0 mg/l of dissolved oxygen to the discharge with typical transfer efficiencies of up to 30%.

Based on the observed performance of the aeration system, a model was developed to estimate the cost of dissolved oxygen mitigation under various scenarios. Based on this model, in a typical year the installed aeration system recovers about 70% of the approximately $350,000 in revenues which would be lost by using wheel gate bypass for dissolved oxygen mitigation. Future enhancements of the system could recover an additional $60,000 per year.

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[1]Engineer, Paul C. Rizzo Associates, Inc., 300 Oxford Drive, Monroeville, PA 15146.

[2]Project Manager, Paul C. Rizzo Associates, Inc., 300 Oxford Drive, Monroeville, PA 15146.

[3]President, Paul C. Rizzo Associates, Inc., 300 Oxford Drive, Monroeville, PA 15146.

INTRODUCTION

The Youghiogheny Lake Dam (Dam) is a 184-foot high, earth-fill dam located on the Youghiogheny River in southwestern Pennsylvania. The dam includes a low level outlet works and an emergency overflow spillway. Discharge has historically been controlled by three sluice gates located near the midpoint of the low level outlet works. The dam was constructed in the 1940's by the federal government and is operated by the U.S. Army Corps of Engineers (Corps) as a multiple-use project. The primary project purpose is flood control with additional uses for low flow augmentation, water quality control, and recreation.

The Youghiogheny River reach downstream of the dam has State-designated uses for "High Quality Waters, Cold Water Fishes." The minimum effluent dissolved oxyygen concentration for this classification per State regulations is 7.0 mg/l. The Youghiogheny Lake (Lake) typically stratifies during the summer season, resulting in low influent dissolved oxygen concentrations for the dam's low level outlet works. However, sluice gate hydraulics and open channel flow conditions downstream of the gates typically allowed for significant discharge aeration. As a result, historic dissolved oxygen concentrations for discharge from the Dam typically exceeded 7.0 mg/l.

The Youghiogheny Hydroelectric Project (Project) was licensed as Project No. 3623 by the Federal Energy Regulatory Commision (FERC) in August 1985 and achieved commercial operation in December 1989. The Project is a 12-MW run-of-the-river facility retrofitted to the existing low level outlet works of the Dam (see Figure 1). Expected generation for the Project is 48 GWH/year. The Project includes a wheel gate control structure at the exit of the outlet works and a penstock branching from the outlet works and feeding two generating units in the Project powerhouse. The FERC license requires that the State-described minimum dissolved dissolved oxygen concentration (i.e. 7.0 mg/l) be maintained in Project discharges. A complete description of the Project has been previously documented (Barton, 1991).

In its typical operating mode, the Project passes all Dam discharge, typically about 800 cfs during the summer, through the turbine units with minimal aeration to discharge. In the event turbine units are not available, or discharge exceeds powerhouse capacity, discharge is bypassed to the Project's wheel gate control structure, thus being well-aerated. Based upon lake stratification observed during the summer months, such bypass discharge (with associated loss in power generation) would be required for sustained periods of time so as to maintain the required dissolved oxygen standard. Based on the first three summers of Project operation, influent dissolved oxygen levels typically drop below 7.0 mg/l from early August to mid-October, reaching a typical minimum of about 2.0 mg/l in mid-September. Water temperatures typically peak at 20°C in mid-September, which corresponds to a saturation dissolved oxygen concentration of about 8.5 mg/l.

INVESTIGATION OF AERATION SYSTEMS

Based upon evaluations conducted during the licensing of the Project, a discharge aeration system was recognized as a possible feature necessary to

comply with the requirements of Article 40 of the Project FERC License. At that time, however, the need for an aeration system was not definite. As such, the Project was not equipped with an operational system during construction. However, prior to final fabrication of turbine components, turbine aeration systems were evaluated and draft tube aeration was concluded to be viable for the Project. As a result, a steel aeration channel was welded to the outside perimeter of each draft tube prior to concrete encasement. These channels are located approximately four feet below the turbine runners. It is noted that aeration ports (i.e. openings in the draft tube walls) were not provided at that time.

Based upon Lake stratification (and associated decreasing influent dissolved oxygen concentrations for the Project) observed during July 1990, the need for an aeration system became apparent. Partial wheel gate bypass was implemented for 94 days during 1990, and in September 1990, due to a combination of low water flows and low influent dissolved oxygen levels (as low as 1.3 mg/l), the generating units had to be completely shut down for 15 days and all flow bypassed. As a result, an investigation of discharge aeration systems was implemented in August 1990 and continued through October 1990. The investigation continued during the stratification period of the summer of 1991. The following sections present the results of the aeration system investigations.

The investigation of aeration systems for the project included evaluation and testing of various mechanical aeration systems including consideration of wheel gate bypass, surface and submerged aerators, turbine venting, high-pressure air injection into turbine draft tubes, and low-pressure air injection to turbine draft tubes. A summary of the results from these investigations is provided as follows:

Wheel Gate Bypass. Aeration of Project discharge by bypassing water through the Project wheel gate commenced in July 1990. As expected, the wheel gate proved to be an effective aerator. However, this significantly reduced the amount of power generated, and so, wheel gate bypass was not considered as a viable means of primary aeration. Partial bypass of flows could be used to augment other aeration systems.

Surface and Submerged Aerators. Two types of commercially available surface aerators where tested in the project tailrace in August 1990. These aerators had a negligible effect on dissolved oxygen concentrations. Aeration by means of a submerged, perforated-pipe aerator (1200 cfm) was tested in the discharge area in July 1991, but this also had a minimal effect on dissolved oxygen concentrations.

Turbine Venting. Turbine venting via the air injection channels available on the Project draft tubes was attempted in August 1990. However, due to site tailwater conditions and turbine characteristics, venting could not be achieved with the Project turbines.

High Pressure Air Injection. Air injection via the air injection channels available on the project draft tubes was evaluated in August 1990. A test program was implemented using on-site and rented air compressors. The test program

included evaluation of varying air volumes up to 450 cfm, varying project discharge rates, and varying air injection port configurations. Based upon these initial evaluations, draft tube aeration was deemed effective for the project. In addition, it was also determined that the injection of low pressure air would be effective for the site.

Low Pressure Air Injection. Based upon the results of the high pressure air injection tests, a permanent low pressure air injection system was designed for the site. Installation of this system, including a 1000 cfm blower for each of the two turbine units, was complete in early September 1990. Based upon testing of the permanent system during September and October 1990, the feasibility of the system was confirmed. Tests of the system continued during the summer of 1991. These tests also evaluated using both installed blowers (i.e. 2000 cfm air) to inject air into a single turbine unit; tests results confirmed the feasibility of this system and indicated that mechanical limitations would likely preclude further additional air injection to a single turbine unit.

DRAFT TUBE AERATION SYSTEM - DESIGN

As noted previously, a permanent draft tube aeration system (i.e. low pressure air injected into a channel located on the turbine draft tubes) has been installed at the project. A schematic of this aeration system is shown on Figure 2. This system includes the following features:

- Two 1000 cfm positive displacement blowers (50 HP,8 psi).
- Steel piping (6-inch diameter) and associated valves and fixtures connecting one blower to each of the Project turbine units through the original air injection channels.
- Flexible rubber hose (6-inch diameter) and associated valves and fixtures allowing connection of the blower for Unit 2 to the draft tube of Unit 1 through one 6-inch diameter hole located just below the original air injection channel.
- Steel channel (8-inch) welded to the outside diameter of the draft tube and embedded in concrete.
- 24 air injection ports (15/16-inch diameter holes) in each turbine unit's original air injection channel.

Flow rates through each generating unit range from 800 to 250 cfs, resulting in air/water contact times from the point of injection to the surface of the Project tailrace of 8.0 to 13.5 seconds, respectively. The draft tube aeration system was installed at a capital cost of about $60,000.

DRAFT TUBE AERATION SYSTEM - PERFORMANCE

Based upon tests completed to date, the efficiency of the installed air injection system (i.e. dissolved oxygen uptake as a fraction of oxygen injected) typically ranges from 17 to 26%. As indicated in Figure 3, lower air/water ratios and lower influent dissolved oxygen concentrations generally result in higher transfer efficiencies.

The amount of dissolved oxygen uptake provided by the aeration system has typically been 2.0 to 5.0 mg/l, depending on several parameters, including air/water ratio, oxygen saturation concentration (around 8.5 mg/l in late summer), and influent dissolved oxygen concentration. The current aeration system is typically able to aerate full discharges (i.e. no flow needs to be bypassed)to the required standard for influent dissolved oxygen concentrations as low as 4.5 mg/l. As the influent levels drop lower, bypass flows must be gradually increased (and hence generating flows decreased) to meet the standard of 7.0 mg/l. Figure 4 shows measured data for air/water ratio and dissolved oxygen uptake for influent dissolved oxygen levels above and below 7.0 mg/l.

The injection of large quantities of air directly below the turbines typically causes a decrease in turbine efficiency. Experienced efficiency losses at the Project related to the aeration system have typically been less than 3.0% (see Figure 5). Thus, the additional generation allowed by the aeration system far outweighs the loss in turbine efficiency.

Tests were conducted on three days in 1992 to compare transfer efficiencies achieved by injecting air through the air injection channels originally installed on the draft tubes with the efficiencies achieved by injecting air through the single hole later installed below the air injection channels. Transfer efficiencies for the air injection channels (24 15/16-inch diameter holes) were 20.4%, 22.8%, and 23.2%, while transfer efficiencies for the single 6-inch diameter hole were 17.4%, 24.3%, and 25.8%.

As indicated in Figures 3 and 4, the performance of the aeration system at the Project was similar to documented performances for systems at seven Tennessee Valley Authority hydroelectric projects (Harshbarger, 1983) and at an Alabama Power hydro project (Miller, 1983). Note that the TVA aeration systems, like Youghiogheny, included the injection of forced air; the Alabama Power aeration system consisted of turbine venting. A model developed by Alabama Power (EPRI, 1990) for predicting dissolved oxygen uptake was used to compare predicted uptakes with actual Project uptakes. This model, however, yielded results which have been generally inconsistent with results measured in the field for the system at the Youghiogheny Hydroelectric Project.

IMPACTS ON POWER GENERATION

Based on the observed performance of the turbines and the draft tube aeration system, historic reservoir discharge and elevation data, and measured influent dissolved oxygen levels, a model was developed to estimate the effects of dissolved oxygen mitigation on Project generation including the following:

- The amount of power which could be generated in an "average" year during the stratification period (July 1 to October 31) if dissolved oxygen mitigation were not required;
- The amount of power which would be lost in an "average" year due to bypassing flows if the aeration system were not installed;
- The amount of this "lost" power which is recovered by the aeration system; and

- The additional benefit which may be gained by puchasing and installing two more blowers.

For this model, average flow rates, lake elevations, and influent dissolved oxygen concentrations were determined for each day of the stratification period. The amount of power which would be generated was then estimated based on the hydrologic conditions each day and observed turbine performance data. When influent dissolved oxygen fell below 7.0 mg/l, the observed performance of the aeration system, including effects on turbine efficiency, was used to determine the flow and blower configuration which resulted in the maximum power generation. The results are summarized in Table 1. Figure 6 compares the daily average potential power generation with the power generation allowed by the current aeration system, as influent dissolved oxygen falls below the required standard.

TABLE 1

AERATION SYSTEM	TOTAL GENERATION (JULY-OCTOBER) (MWH)	BLOWER POWER CONSUMPTION (MWH)	POTENTIAL GENERATION LOST
Aeration Not Required	16,080	-	-
Wheel Gate Bypass	8,520	-	47%
Two Blowers (current)	13,920	147 (1.1%)	14%
Four Blowers (future)	15,120	206 (1.4%)	7%

As indicated in Table 1, the existing draft tube aeration system recovers about 70 percent of the power generation (about $270,000 per year, assuming $0.05/KWH) that would otherwise be lost due to dissolved oxygen mitigation by wheel gate bypass, at an annual operational cost of about $7,000. The addition of two more blowers could recover another $60,000 per year in lost revenue, at an additional operating cost of less than $3,000.

CONCLUSION

Due to Lake stratification and subsequent low dissolved oxygen concentrations in the Youghiogheny Hydroelectric Project discharges, several aeration methods were investigated and ultimately a low pressure draft tube air injection system was installed. This system has successfully injected up to 5.0 mg/l of dissolved oxygen to the Project discharge, and thus has allowed for compliance with the water quality requirements of the 401 Permit, while maintaining near maximum power generation during the summertime stratification period.

The performance of the aeration system at the Project has resembled documented typical performances at other hydro plants, including seven TVA projects which implement forced air injection and an Alabama Power hydro plant which implements turbine venting.

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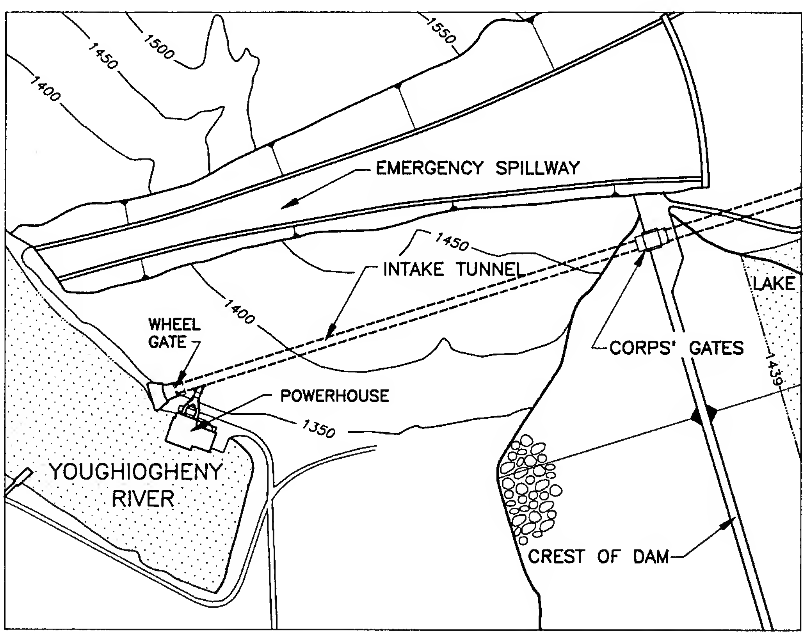

FIGURE 1. PLAN VIEW OF YOUGHIOGHENY HYDROELECTRIC PROJECT.

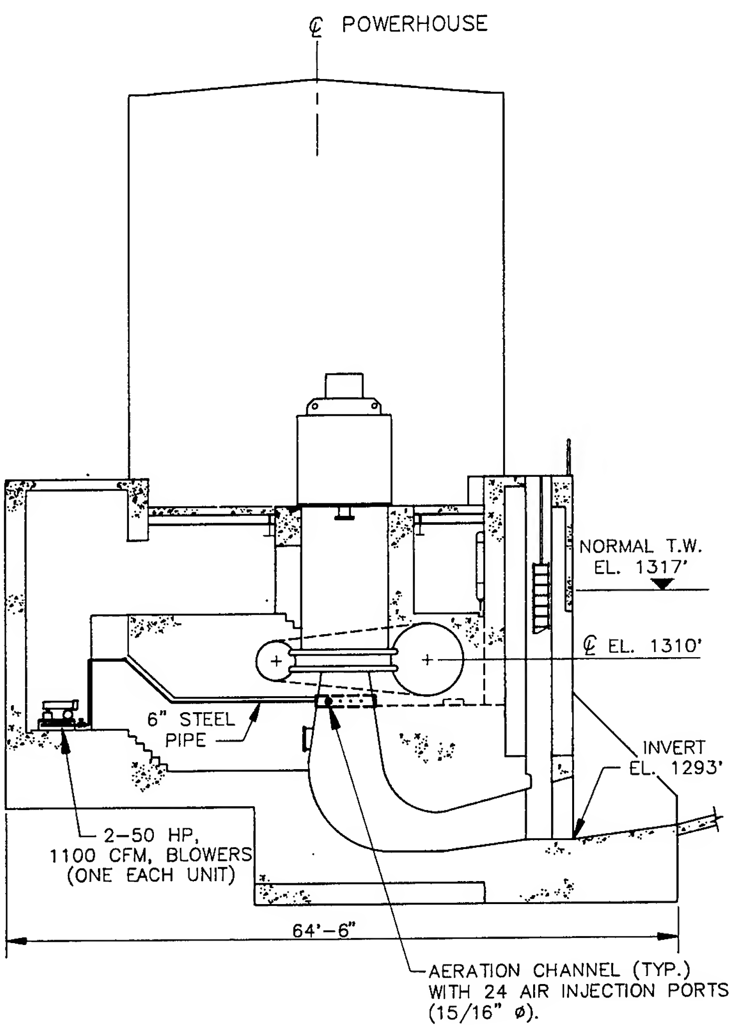

FIGURE 2. SCHEMATIC OF DRAFT TUBE AERATION SYSTEM.

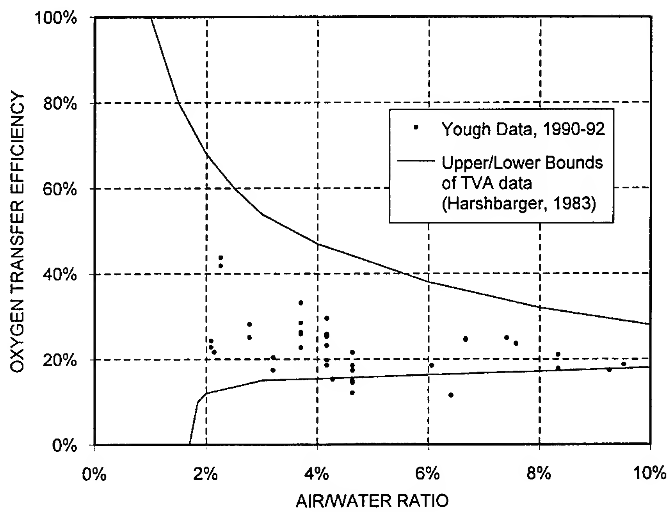

FIGURE 3. AIR/WATER RATIO VS. OXYGEN TRANSFER EFFICIENCY.

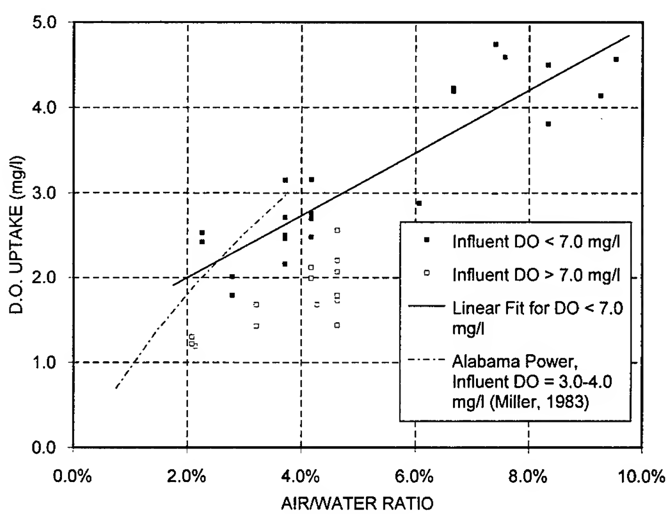

FIGURE 4. AIR/WATER RATIO VS. DISSOLVED OXYGEN UPTAKE.

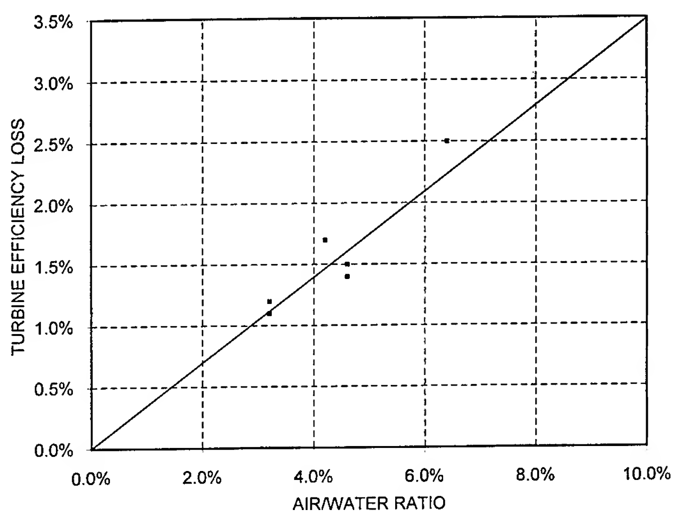

FIGURE 5. TURBINE EFFICIENCY LOSS AT VARIOUS AIR/WATER RATIOS.

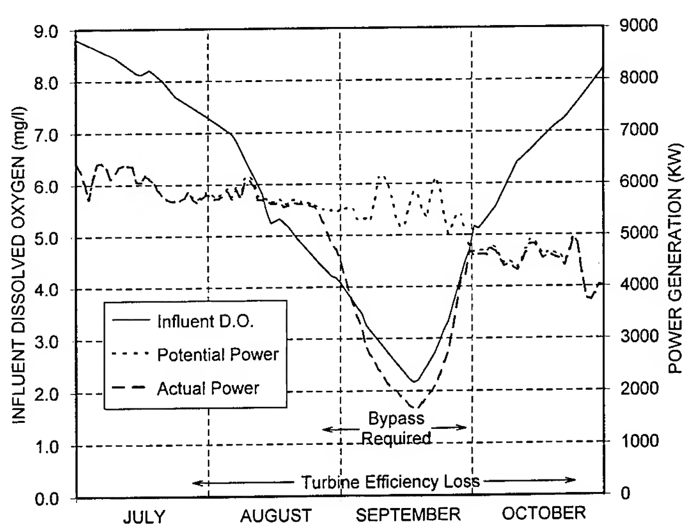

FIGURE 6. TYPICAL POWER GENERATION WITH/WITHOUT DO MITIGATION.

NEPA and License Renewal: A Case Study of the EIS Process for Projects in Relicensing

Gary D. Bachman
Mary Jane Graham[1/]

I. Abstract

Many of the hundreds of hydropower projects facing relicensing by the Federal Energy Regulatory Commission (FERC) over the next few years can expect the scrutiny of an environmental impact statement (EIS) pursuant to the National Environmental Policy Act (NEPA). This paper explores a number of issues raised by applying the EIS process to a project in relicensing, and places them in context by discussing a case in which they have already arisen, the relicensing of the Kingsley Dam Project No. 1417 on the Platte River in Nebraska.

II. Background

FERC has determined that relicensing proceedings are subject to the threshold review requirements of NEPA, with the level of analysis to be determined on a case by case basis.[2/] If relicensing is found to be a "major federal action" that would "significantly affect[ ] the quality of the human environment," alternatives must be examined in an EIS.[3/] FERC has already decided to develop an EIS in several relicensing proceedings, including that for the Kingsley Dam Project. Other projects can expect the same scrutiny. In

1/ Gary D. Bachman is a Member and Mary Jane Graham is an Associate of the law firm of Van Ness, Feldman & Curtis, A Professional Corporation, 1050 Thomas Jefferson Street, N.W., Washington, D.C. 20007.

2/ 18 C.F.R. § 380.5(b)(9).

3/ 42 U.S.C. §§ 4332(2)(C)(iii), (E) (1988); 40 C.F.R. § 1502.14.

determining what role to play in the EIS process, licensees must recognize that even in the most contentious contested hearings, FERC rules and regulations are biased against holding an evidentiary hearing on the merits of a license renewal; FERC's position is that a "paper hearing" is sufficient in virtually any circumstance. Because the public comment and documentation provisions of the EIS process are extensive, and can be expected to explore all contested issues, the EIS and supporting documentation will likely comprise the record on which FERC makes its final decision. The EIS may be the "paper hearing" on relicensing; if so, the comment process will likely provide the licensee's only opportunity to challenge the factual assertions of others and to make its case.

Regulations[4/] require an EIS to analyze a range of alternatives to be considered in the ultimate decision on relicensing.[5/] NEPA is procedural, however, designed solely to place information before the decision-maker, so the EIS will not dictate any outcome.[6/] FERC initiates the process by inviting comments on the scope of the EIS. A Draft EIS (DEIS) is then prepared for comment, followed by a Final EIS (FEIS). As soon as possible, the "agency's preferred alternative" (FERC Staff's view at the time of the alternative which should be adopted) must be identified from among the alternatives examined.[7/] As FERC Staff must consider other statutory charges, the "agency's preferred alternative" may or may not be what is "best" for the environment. The EIS process concludes with the Commissioners' published final order, the Record of Decision (ROD). An ROD must list the alternatives considered and identify both the one adopted and the "environmentally preferred alternative."[8/] They need not and likely will not be the same, given the other factors to be weighed under the Federal Power Act (FPA).

The EIS must be integrated into already complex consultation and decision-making processes. The Electric Consumers Protection Act (ECPA) revised the FPA to require FERC to seek recommendations from resource

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4/ FERC has adopted the Council for Environmental Quality's (CEQ's) generic regulations for implementing NEPA except where they conflict with other statutory requirements. 18 C.F.R. §§ 380.1-380.14 (1992).

5/ 40 C.F.R. § 1502.14.

6/ Vermont Yankee Nuclear Power Corp. v. Natural Resources Defense Council, 435 U.S. 519, 558 (1978); Robertson v. Methow Valley Citizens Council, 109 S. Ct. 1835, 1846 (1989).

7/ 40 C.F.R. § 1502.14.

8/ 40 C.F.R. § 1505.2.

agencies regarding potential license conditions to benefit wildlife,[9] which must be worked into the alternatives analyzed in the EIS if they can reasonably be carried out. In addition, if endangered species are potentially implicated, further consultation is required under section 7 of the Endangered Species Act (ESA).[10] Steps identified by the Department of Interior (DOI) as needed to avoid jeopardy to endangered species must also be examined in the EIS before they can be implemented.

The relicensing of the Kingsley Dam Project has been mired in the EIS process for some time. Section 10(j) recommendations were provided to FERC in November 1990. In January 1992 a DEIS was issued, but after receipt of substantial criticism from many participants, FERC announced that it plans to prepare a revised DEIS. Public meetings on the scope of that document were held in December 1992, with written comments filed by the participants in February 1993. A revised DEIS is expected in the fall of 1993. Formal consultation under the ESA will follow the revised DEIS; DOI has declined to consult formally before that time.

III. The Scope of the EIS: What Alternatives Must Be Explored for a Hydro Relicensing?

The regulations implementing NEPA expressly provide that in an EIS an agency need only explore "reasonable" alternatives and a "no action" alternative.[11] These bounds on the required scope of an EIS reflect the fact that NEPA is procedural, not outcome-determinative. The information made available through the EIS focuses on what the decision-maker can actually use or adopt; the EIS is not to be an academic exercise. The requirements of ECPA and the ESA do not alter these bounds on the EIS. If section 10(j) recommendations are reasonable they must be included in the EIS under NEPA.[12] If they are not reasonable, they are not consistent with the FPA and may be rejected by FERC;[13] FERC need not explore them in an EIS before doing so. Similarly, if DOI identifies measures which it asserts are needed to avoid jeopardy to an endangered species but which cannot reasonably be carried out, other processes must be invoked; analyzing these measures in an EIS will not make them feasible.

9/ 16 U.S.C. § 803(j) (1988).

10/ 16 U.S.C. § 1536 (1988).

11/ 40 C.F.R. § 1502.14.

12/ 40 C.F.R. § 1502.14(f).

13/ 16 U.S.C. § 803(j)(2).

A. What Are Reasonable Alternatives?

For an alternative to be "reasonable," it must satisfy two tests for inclusion in an EIS. First, it must be practical and feasible technically, economically and using common sense.[14] Conjectural possibilities which are unlikely to be implemented need not be included.[15] There is no bright line between the feasible and the infeasible; reasonable alternatives need not be entirely within the authority of FERC or of the licensee to implement.[16] It is axiomatic, however, that if an alternative includes environmental enhancement programs which would bankrupt the licensee, it is not reasonable. Similarly, an alternative cannot be feasible if it would require a major restructuring of state water law or would preclude irrigated agriculture in a region.

The second test for reasonableness is the ability of the alternative to fulfill the primary purpose of a project.[17] Courts have found that if the principal purpose of a hydro project is something other than energy production, no alternative need be considered in an EIS which does not fulfill that function.[18] For a project like Kingsley Dam that primarily provides irrigation, any alternative that seriously impinges on the ability of the reservoir to serve irrigators is unreasonable per se.

Some participants in the Kingsley Dam proceedings have urged FERC to adopt a far different interpretation of the range of alternatives in an EIS. They seek inclusion of unachievable section 10(j) recommendations and a "maximum wildlife benefits" alternative that would run the water project to benefit riparian wildlife downstream, allowing only incidental water use by others. Although

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14/ Vermont Yankee Nuclear Power Corp., 435 U.S. at 551; CEQ, Forty Most Asked Questions Concerning CEQ's National Environmental Policy Act Regulations, 46 Fed. Reg. 18,026, 18,027 (1981) (Question 2(a)).

15/ CEQ, Guidance Regarding NEPA Regulations, 48 Fed. Reg. 34,263, 34,267 (1983); e.g., Roosevelt Campobello Int'l Park Comm'n v. EPA, 684 F.2d 1041 (1st Cir. 1982).

16/ See 40 C.F.R. § 1502.14(c); 46 Fed. Reg. at 18,027 (Question 2(b)).

17/ 48 Fed. Reg. at 34,267; e.g., National Wildlife Fed'n v. FERC, 912 F.2d 1471, 1484-85 (D.C. Cir. 1990); Northwest Coalition for Alternatives to Pesticides v. Lyng, 844 F.2d 588, 594 (9th Cir. 1988); City of Angoon v. Hodel, 803 F.2d 1016, 1021 (9th Cir. 1986), cert. denied, 484 U.S. 870 (1987); Roosevelt Campobello Int'l Park Comm'n, 684 F.2d at 1047; Natural Resources Defense Council v. SEC, 606 F.2d 1031, 1054 (D.C. Cir. 1979).

18/ National Wildlife Fed'n v. FERC, 912 F.2d at 1484-85.

their advocates half-heartedly assert these alternatives are reasonable, they largely brush aside the reasonableness standard. In their principal argument for including a "maximum wildlife benefits" alternative, these participants assert that for FERC to integrate the EIS process and the FPA, it must examine a "maximum wildlife alternative" as the "mirror image" to current operations -- which they claim "maximize" developmental values. Based on ECPA's charge that FERC give "equal consideration" to developmental and non-developmental values, they argue that FERC must identify the two extremes and adopt license conditions which split the difference. They are incorrect; that provision, like NEPA, was intended primarily to increase awareness of environmental concerns.[19/] "Equal consideration" in no way requires "equal treatment" or "equal weight" for the values balanced in determining what is in the public interest.[20/] In addition, no court interpreting ECPA has ever suggested a need for detailed analysis of artificial or extreme proposals in an EIS; case law in fact indicates that ECPA imposes no duty to perform any further analysis.[21/]

The "maximum wildlife benefits" alternative is a politically motivated attempt to shift the perceived middle ground. By radically expanding the range of alternatives considered in the EIS in a single direction, the alternative's proponents make the devastating impacts of their real agendas seem more moderate. In addition, the "mirror image" characterization presents current operations as equally extreme and unreasonable, encouraging FERC to distance itself from current practices in selecting its preferred alternative. In fact, current operations at Kingsley Dam are far from "maximizing" developmental values; the licensee contributes extensively to recreation and wildlife values in various ways including maintaining instream flows for wildlife.

FERC appears to be of mixed mind: it stated it would analyze the "maximum wildlife benefits" alternative only as "consistent with the degree to which it is determined to be 'reasonable,'" but also noted that analysis of such an alternative would be "useful." The initial DEIS did not explicitly discuss reasonableness, although it declined to examine in detail alternatives and enhancement programs which were too burdensome financially for the licensee. Now, although FERC has indicated that it is expanding the range of alternatives considered in the revised DEIS, it has not stated how it will define or treat suggested alternatives which are unreasonable.

19/ United States Dept. of the Interior v. FERC, 952 F.2d 538, 545 (D.C. Cir. 1992).

20/ National Wildlife Fed'n v. FERC, 912 F.2d at 1481; California *ex. rel.* State Water Resources Control Bd. v. FERC, 966 F.2d 1541, 1550 (9th Cir. 1992).

21/ E.g., Friends of the Ompompanoosuc v. FERC, 968 F.2d 1549 (2d Cir. 1992).

B. What Is the "No Action" Alternative?

The "no action" alternative must be included in the EIS, whether or not it is a "reasonable" alternative. If unreasonable, it should be analyzed in the EIS only as necessary to fulfill its primary function: when other reasonable alternatives are analyzed, the "no action" alternative provides the baseline by which their costs and impacts are measured. For a "green field" project, the "no action" alternative is easy to define -- the project is not built and the status quo continues. For an existing project, the definition is less obvious. Is it a continuation of current operations? Denial of the new license with removal of the project? Denial of the new license with non-hydro functions continuing? Simulating "green field" conditions by predicting what might exist if the project had never been built?

FERC has repeatedly taken the position that the appropriate choice is a continuation of current operations, and is supported by regulatory guidance and case law. "[W]here ongoing programs initiated under existing legislation and regulations will continue, even as new plans are developed..., the 'no action' alternative may be thought of in terms of continuing with the present course of action until that action is changed. Consequently, projected impacts of alternative management schemes would be compared in the EIS to those impacts projected for the existing plan."[22] This in fact closely parallels what would happen if FERC truly took no action on a relicensing application; annual licenses would issue indefinitely.[23]

FERC used current operations to develop the "no action" alternative or "baseline" in the initial DEIS for the Kingsley Dam Project, and affirms that despite arguments for other interpretations, it will again in the revised DEIS. Intervenors and resource agencies including DOI continue to assert that the "no action" alternative or "baseline" must be either a simulated "green field" in which the project had never been built, or denial of the new license and removal of the project. The arguments offered in support of these are contorted or nonexistent and ignore fundamental logic: the project exists and extensive effort would be needed to remove it. That effort cannot be described as taking "no action."

There is a reason that these participants in the Kingsley Dam relicensing proceeding are fighting hard to defend improbable positions: perception. Because the "no action" alternative is used to identify the impacts of other alternatives, if it were defined without the project and its irrigators, every alternative in the EIS would be attributed with virtually every change in the

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22/ 46 Fed. Reg. at 18,027 (Question 3).

23/ 16 U.S.C. § 808(a) (1988).

region over the past fifty years. In the EIS, the reasonable alternatives would stand in the same order relative to one another in what they achieve. But when FERC turns to balancing interests under the FPA, extensive and expensive environmental programs may be weighed differently if viewed as enhancing current conditions rather than mitigating -- or not quite mitigating -- an alternative's perceived adverse impacts. As with the "maximum wildlife benefits" alternative, this is an attempt to shift the point at which FERC strikes its balance.

IV. <u>The Analysis of Alternatives: How does FERC Compare its Options?</u>

Regulations state that the heart of an EIS is the comparison of alternatives: an EIS is to present the impacts of each in a comparative form which will sharply define the issues and provide a clear basis for choice by the decision-maker.[24/] Agencies have the discretion to use their expertise in determining how to fulfill this obligation. In the initial DEIS for the Kingsley Dam Project, FERC directly compared alternatives in two ways, by assessing the financial costs of programs compared to the "no action" alternative, and through "value function indices" developed for selected flow-related parameters. These practices make FERC's comparisons of alternatives questionable because they leave so much out of the comparative process. Yet FERC appears poised to use them again, not only in the revised DEIS for the Kingsley Dam Project, but in other relicensing proceedings to come.

A. <u>What Costs Must Be Compared?</u>

It appears self-evident that in an EIS FERC must examine all of the costs and all of the benefits of each alternative in order to determine the relative advantages and disadvantages of each. In the initial DEIS for the Kingsley Dam Project, however, FERC attempted to simplify the task by normalizing the alternatives: all flow regimes were adjusted to have the same non-power operational requirements, and all enhancement plans were assumed to include the existing programs in the "no action" alternative. With underlying costs forcibly equalized, flow and non-flow enhancements were viewed as merely additive, and FERC's assessment of the cost of alternatives was based on the monetary costs to the project of the additions. This approach was widely criticized both for failing to recognize the uniqueness of each alternative, and for failing to take into account the costs which had been normalized out of the comparison.

FERC has stated that in the revised DEIS, it intends to analyze each proposal as a whole, without making normalizing adjustments, but it has not

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24/ 40 C.F.R. § 1502.14.

described how it will make its comparison of costs. Because resources are limited, the various types of costs and benefits are inextricably interrelated: all operational changes will have trade-offs. Since underlying costs will no longer be fixed, only the total costs of each alternative appear capable of comparison to one another and to whatever cap on costs may eventually be found to be the outside bound of reasonableness.[25] Both the total cost FERC ascribes to each alternative and the cap placed on costs must include non-monetary and monetary costs, as well as recognizing costs paid by entities other than the licensee. But not all costs can be reduced to dollars or some other common denominator. Even after FERC makes every effort to capture the monetary and non-monetary costs of each alternative, real costs will have been missed. Before selecting the alternative it believes represents the best balance, FERC will need to identify and afford some weight to costs which can be expressed only qualitatively, such as flexibility in meeting goals.

B. How Are Value Functions Used?

The Kingsley Dam Project has many contested issues of fact about which mountains of data and analysis have been filed with FERC. In the initial DEIS, FERC tried to simplify its consideration of these issues by selecting ten or fifteen "key" flow related parameters such as availability of habitat for a particular species, and forcing them into normalized value functions. Most value functions represented FERC's view of the value on a scale of 0 to 1.0 of a single parameter as a function of flow rate or lake level, but some were apparently weighted combinations of several related parameters. It was completely unclear in the initial DEIS how these functions or the value function indices they yielded for each individual alternatives were used in balancing interests to select the "agency's preferred alternative."

Over the objections of virtually every participant in this proceeding, irrespective of interest or viewpoint, FERC has unfortunately affirmed that it will continue to use value functions in its revised EIS. In dismissing criticisms of the value function approach, FERC described value functions as merely "summary indicators of flow-related changes to important resources." As with the initial DEIS, FERC Staff is maintaining secrecy as to how it intends to use these "summary indicators" in selecting the "agency's preferred alternative," and how, if at all, the decision-making process might include the many costs and benefits not reflected in any of the value functions.

25/ This does not mean FERC could not reject an alternative as unreasonable without totalling up all costs; if one aspect of an alternative would make it infeasible, other aspects could not make it otherwise.

There are a number of reasons the value function approach is an inaccurate and incomplete means of identifying strengths and weaknesses. Perhaps most importantly, value functions create the appearance of objectivity while actually introducing hidden subjective judgment. Placing the value functions on a normalized scale gives the impression that the "best" flow regime can be identified by selecting the one with the highest overall total or weighted average of value functions. In theory, if the value functions are properly selected and truly reflect the trade-offs and compromises that must be made, the alternative reflecting the best balance of "key" indicators will have no particularly high values and no particularly low ones. In practice, by focusing on only a few selected parts of the bigger picture, any error in how "key" parameters are selected and modeled can control the outcome of the assessment. The selection of parameters considered "summary indicators," the way the scale of each value function was defined and the weight each is given relative to the others all reflect subjective choices made by FERC which to date have not been suject to review. It would be very misleading under such circumstances to pick an alternative which scores the most points as "preferred."

The dangers of the apparent objectivity of the value function approach may be compounded when FERC seeks to balance costs and benefits under the FPA. These pseudo-scientific numerical value function indices may be given undue weight when combined with other costs and benefits which are merely described qualitatively. It appears far more useful to look at the comparative flow duration curves on which many of the value functions are based; the trade-offs between species are more visible, as are the choices made about how to distribute the effects of water shortage among competing users.

V. Recommending an Alternative to Adopt: How Will FERC Select the "Agency's Preferred Alternative"?

The "agency's preferred alternative" in the EIS is the FERC Staff's recommendation to the Commissioners on how the balance of interests under the FPA should be struck. Because an evidentiary hearing on the merits is extremely unlikely, the "agency's preferred alternative" in the EIS may well become FERC's final position with little further discussion. With the EIS' focus on identifying environmental impacts, it is important to use the EIS commenting process to force FERC to acknowledge how it considered other factors. When the Commission makes its final decision on balancing of interests, it cannot be allowed to simply assume that the Staff took these factors satisfactorily into account.

An essential part of balancing should be determining a licensee's fair share of desired enhancement programs or efforts to mitigate past developmental impacts. Although the EIS should include any otherwise reasonable alternatives within the licensee's ability to pay, FERC must also consider how the value and

costs of the project are to be shared. Requiring environmental enhancement up to the ability to pay means that ratepayers, irrigators or whoever built and is the principal beneficiary of a project, will no longer receive any value for investment in and operation of the project. FERC should be explicitly invited to state if that is its policy under the FPA. In addition, the role of others in creating conditions of concern to environmental interests should be part of determining a fair share to pay; for example, development upstream and in the area around the project has played a far larger role than the Kingsley Dam in changing conditions in the Platte River. A licensee should not have to shoulder the full burden of addressing such conditions, just because it is the furthest downstream or because FERC has jurisdiction over it.

Other policy decisions should also frame what FERC adopts as the "agency's preferred alternative." Like the question of fair share, they may lead to the selection of an alternative other than what is "environmentally preferred." FERC should be asked to determine what type of enhancements are appropriate or preferred as a matter of policy under the FPA. For example, should they be tailored to past impacts or should they be those that provide the most positive impact for the resources expended? Should efforts be made to avoid adverse impacts on entities other than the licensee, even if blind cost/benefit analysis would lead to a different conclusion? Should FERC encourage better stewardship of land and water by licensees not yet facing relicensing by recognizing or giving additional weight to existing programs?

FERC has not yet come to grips with questions such as these in the EIS process for the Kingsley Dam. In the initial DEIS for the Kingsley Dam Project, it was not clear how FERC justified its selection; the value functions indices, tabulated monetary costs and identified non-monetary costs for each alternative were never collectively described in terms of balancing interests under the FPA. Nor has FERC disclosed what approach it will take to selecting the "agency's preferred alternative" in the revised DEIS. The licensee in this case has put balancing issues before FERC already, even though it is still fairly early in the EIS process, and final balancing interests under the FPA is the purview of the Commissioners and will not be completed until the ROD. In light of the almost certain inavailability of an evidentiary hearing on the merits, the EIS process appears the best and perhaps the only opportunity for the licensee to make its case on the balance of interests.

SNAKE RESERVOIR DRAWDOWN
A PROGRESS REPORT

John J. Pizzimenti[1], Kevin Malone[1], Paul Tappel[2] and Brian Sadden[2]

Abstract

The Snake River Sockeye Salmon and three other Columbia River salmon stocks that run past eight federal dams (about 10,000 MW aggregate capacity) were officially listed as threatened or endangered by the National Marine Fishery Service (NMFS) in 1992. In order to preserve, and hopefully to restore these fisheries, the Pacific Northwest Power Planning Council (Council), Bonneville Power Administration (BPA), the U.S. Army Corps of Engineers (Corps), plus eight other state, federal, and tribal representatives formed the Snake River Drawdown Committee (Committee). The Committee's objective is to evaluate the engineering feasibility and biological efficacy of lowering five hydroelectric project reservoirs (four on the Snake and one on the Columbia) to increase smolt survival during their outmigration to the ocean in April - June of each year. The Committee will report its findings to the Council in the Fall of 1993. The purpose of this paper is to review the issues and communicate the progress of the Committee and its consultant, Harza Northwest (Harza).

Nine options are being studied by the Corps, from a 33-foot Drawdown of four Snake River reservoirs to complete (100-foot) Drawdown of four Snake River reservoirs plus partial Drawdown of John Day. Cost estimates range from two to six billion dollars, and schedules span up to seventeen years to complete the project.

Harza has offered specific suggestions to reduce both costs and schedule of the Corps' designs including use of the existing dams as upstream coffers to construct new low level spillway outlet structures. Additionally, Harza has come up with several alternative proposals that could improve fish passage around the dams that may merit study. These include side channel spillways and a downstream weir. We also have reviewed the biological literature which indicates that although Drawdown may help salmon, there are numerous causes of mortality that must also be addressed

[1]Harza Northwest, Inc., 9600 Southwest Oak, Suite 350, Portland, Oregon 97223
[2]Harza Northwest, Inc., 2353-130th Northeast, Suite 200, Bellevue, Washington 98005

in all stages of the life cycle if we expect to increase the number of adult fish returning. Harza has proposed specific data that should be collected to increase confidence about any changes that should be made to the system, including Drawdown.

Introduction

Harza Northwest engineers and scientists were asked to provide a third-party review of the U.S. Army Corps of Engineers plans for "Drawdown." Drawdown is a reduction in the pool elevation of four mainstem Snake River dams plus the John Day pool on the Columbia. Inspired by drought and declining salmon stocks (one endangered and three threatened), Drawdown is proposed to reduce the cross-sectional area of reservoirs and increase hydraulic velocity. The objective is to accelerate out migrating Snake River juvenile salmon.

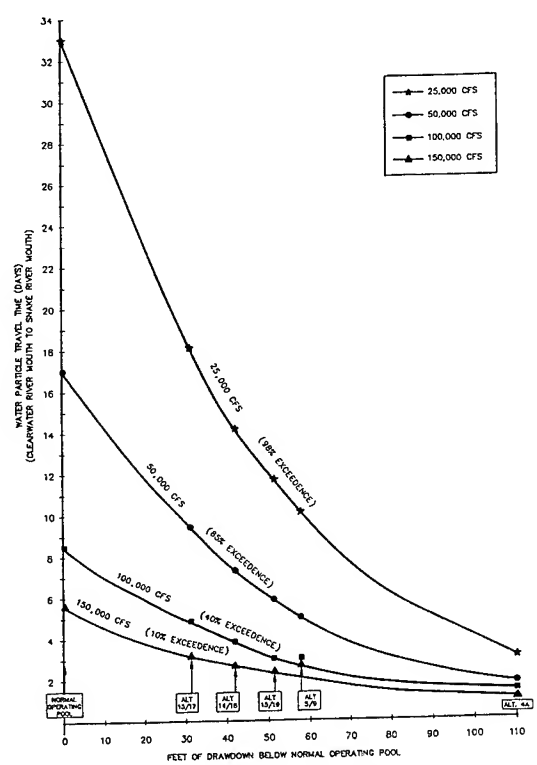

FIGURE 1. WATER PARTICLE TRAVEL THROUGH SNAKE RIVER PROJECTS, COMPARED TO DRAWDOWN ALTERNATIVES

By the use of a Drawdown Test in March 1992 and subsequent modeling efforts, the Corps has demonstrated that Drawdown will increase average water particle velocity. For example, a Drawdown of 33 feet would shorten the average water particle travel time (WPTT) in the Snake reservoirs by about five days at 75 kcfs (Figure 1). If fish traveled at approximately the same speed as water, such efforts would shorten the Snake Reservoir travel by about forty percent. Complete reservoir removal would make more significant changes to WPTT and this is one of the nine options the Corps is studying.

Cost of Drawdown will be significant in both capital costs to redesign the dams ($2 to 6 billion) as well as operational costs related to lost hydropower, navigation, recreation, irrigation and flood control benefits. Additionally, there are numerous engineering and biological problems that must be overcome to allow it to be done safely and in a biologically prudent manner. One necessity is the redesign of fish passage facilities for downstream juvenile migrants and upstream migrating adults. Evacuation of the reservoirs in March using both turbines and possibly spill, is necessary to anticipate arrival of migrating smolts in April. Nitrogen supersaturation could pose a serious problem during spill (Boyer, 1974). As a result, the benefits to the salmon fishery must be predictable, not just in more rapid transit to the estuary, but in true reversal of decline of wild stocks if the investment in Drawdown is to pay off. Finally, the Corps estimates that full Drawdown could take between 12 and 17 years to be fully implemented.

Drawdown is intended to compensate for drought, increasing consumptive uses, and storage. Although delay of migration may be a source of salmon mortality, other factors that are also significant causes of mortality, with or without Drawdown, must be correctly quantified, assigned priority and abated. In order to assess the potential benefits of Drawdown, it is first necessary to understand the causes of mortality of both juveniles and adults. For it will do little good to increase migration speed, only to see whatever benefits Drawdown may provide nullified by other causes of mortality. These causes of mortality are discussed in the context of both existing operations as well as proposed Drawdown condition.

Causes of Fish Mortality in the System

Turbine Mortality. Turbines are estimated to kill between 10% and 15% at each dam of those fish not removed by screened bypass systems. The efficiency of removing fish from the river is about 60% at best, thus 40% or more pass through the turbines at most dams. New fish bypass facilities at Bonneville Dam (Ferguson, 1991) showed very disappointing results in that mortality appeared to be higher for fish utilizing the bypass than those that slipped past the fish screens and through the turbines (22% Vs 16%). Drawdown itself does not address turbine mortality; it only speeds fish toward the dams. Once there, the passage around the dam and turbines must still be addressed. If fish are passed through the turbines under Drawdown it is likely that fish guidance efficiency (FGE) and turbine efficiency will both decrease leading to even higher rates of juvenile mortality. Thus Drawdown, while speeding fish toward the estuary faster, could increase mortality in route unless juveniles are kept out of the turbines. Harza is recommending use of new tagging technology (Hi-Z Turbine Tags) to evaluate turbine mortalities under varying degrees of Drawdown.

Delay. Although slow velocities in reservoirs may cause delay of migration, exactly how and where fish are delayed is not clear. Other than full pool Drawdown, juvenile fish may still encounter delays when they reach the dams because of the current existing designs of fish passage facilities at most of the dams. This is because juvenile fish are surface-oriented while turbine intakes and juvenile bypass systems are located deep in the reservoir. Inability to find the exits, or reluctance to dive deeply may be a significant or even a primary cause of juvenile delay at the dams. Although

circumstantial evidence suggests fish may be delayed at both the dams and in the reservoirs, a clear picture is currently lacking. Harza is recommending specific studies to answer these questions before final judgments are made on the both the design of Drawdown and the biological efficacy of Drawdown.

In regard to passage around mainstem dams, we do know that dams which utilize top spill to pass fish, such as Wells Dam on the Columbia and Ice Harbor on the Snake, may be far more successful in passing large numbers of juveniles quickly and safely away from the turbines and past the dam. Wells Dam passes up to 95% to 97% of the fish utilizing top spill of less than 1% of the total discharge past the dam (Kudera, *et al.*, 1991). Ice Harbor dam currently has no juvenile bypass system, yet 50% of the juvenile fish are estimated to pass through the surface oriented ice and trash sluiceway within only a small fraction of the water (3000-5000 cfs) that passes through the dam. Ice Harbor, like Wells Dam, is using a surface oriented system to temporarily pass juvenile fish until the new turbine bypass system is completed. As one of our tasks for the Committee, Harza Northwest is investigating alternatives that would allow surface oriented juvenile bypass during Drawdown as well as during normal pool operations.

Predation. Predation of juvenile salmon by squawfish and other adult resident species is a known cause of mortality in the Snake and Columbia rivers (T. Poe *et al.*, 1991). Reiman *et al.*, (1991) estimate mean annual loss in John Day Reservoir to be 2.7 million juvenile salmonids. Predation rates increase with the season as a function of increasing water temperature and metabolic activity. The reservoirs, turbines, and bypass outfalls may have inadvertently improved conditions for predation to occur by providing a concentrated stream of stunned or disoriented juvenile salmon in a hydraulically friendly environment to squawfish (Sims and Johnsen, 1977). Predation losses may be estimated between 9-19% in John Day but may be different in other parts of the system (Reiman *op. cit.*). D. Bennett (pers. comm., 1992) estimates at Lower Granite pool that predation losses may be closer to 5% of juveniles passing through the reservoir. It has been hypothesized that Drawdown could reduce predation because of reduced exposure to predators. The counter argument is that predators and prey will be more concentrated in smaller reservoirs and counterbalance any benefits of reduced exposure. Both arguments are oversimplifications of potentially complex predator prey relationships that may occur under Drawdown. For example, Bennett (pers. comm., 1992) observed that stomachs of smallmouth bass contained large numbers of Chinook salmon juveniles after the Drawdown Test of 1992. Normally, Bennett observed that smallmouth bass prey on crayfish, not salmon in Lower Granite studies. Large numbers of crayfish were killed during the Drawdown test. Bennett explains the prey shift as possibly a reflection of change in food availability. Most experts agree that the concentration of juveniles at the outfalls of dams is a major source of the predation problem. Predator densities are six to thirty times higher near the dams. Predation rates in these areas are 50 times higher, per kilometer, than other parts of the system (*op. cit.*).

Mortality from predation or disease may be significant in all parts of the system, not just the reservoirs. Thus Drawdown may not significantly reduce total load on the juvenile populations. Current data indicate that many juvenile fish of hatchery origin in the upper Snake are disappearing

long before they ever reach the first reservoir (Lower Granite). We know this because 14 million juvenile chinook salmon are released from hatcheries in the upper Snake river. Of these 14 million releases, less than 5 million reach the first dam. In recent mark/recapture experiments (data collected by Buettner and Nelson, 1990), it appears that survival between the head of Granite Reservoir and the dam is between 75 and 90% for certain stocks. This is based on downstream recoveries of marked fish released by Buettner at the head of Lower Granite Reservoir. If Buettner's data are indicative, we should be seeing between 9 and 12 million juvenile chinook salmon at the dam if mortality is only 10 to 25% in the reservoir. The data suggest that as many, or even more juveniles may be disappearing before they reach Granite Reservoir. These data present a caveat. And that is, even assuming that Drawdown will help reduce mortality, we should be cautious as to exactly how much of the mortality Drawdown can help. If mortality in reservoir is as little as 10%, Drawdown could be a very expensive fix for a small part of the problem. If it is 25% or higher, it is more attractive.

Hatchery Practice. Currently, hatcheries are accounting for nearly 95% of the return of adult salmonids in the Columbia and Snake rivers. Nearly 30 million hatchery fish are introduced into the Snake River each spring. And nearly 300 million in the Columbia system as a whole. Knowing that Drawdown is going to be very expensive, we need to know whether Drawdown is primarily intended to help hatchery fish or wild fish. The obvious answer is both. Unfortunately, most of what we assume about wild fish behavior is based on hatchery practice and hatchery fish. Hatchery fish are introduced to the river and their migratory timing is affected by hatchery practice. Wild fish may consist of highly variable stocks migrating over much protracted time frames. Wild fish may be suffering from competition and disease brought on by the introduction of numerically overwhelming numbers of hatchery fish. Why not minimize some of these problems by controlling hatchery fish from their very origin. We should learn as much as possible about wild fish, for they are the unique genetic legacy the Endangered Species Act (ESA) seeks to protect and that Drawdown might aid. For the first time, NMFS is proposing to mark all Snake River hatchery fish in 1993 so that wild fish can be identified. Harza Northwest biologists are endorsing specific study plans for mark-recapture experiments of wild fish in 1993. Specifically, we recommend expansion of the number of PIT-tagged fish to evaluate travel-time and survival through Lower Granite Reservoir at normal pool in 1993.

Competition and Density Dependent Mortality. The biological environment of the Columbia and Snake Rivers has been changing continuously for the past 100 years. The most significant physical changes are the construction of dams, however other changes also may be having significant effects. Water quality and the conditions in the Columbia estuary have been degraded. In addition to salmon, there are millions of adult shad and hundreds of millions of juvenile shad that now exist and compete for space and food in the system. Other species of warm and cold water fish have been introduced (smallmouth bass and channel catfish are only two of many species); and hatcheries are pouring in as many smolts as may have ever been produced naturally by the system. If we are exceeding the biological carrying capacity of the river or the estuary , it may matter little how

many smolts we add or save with upstream efforts. Simply stated, "more" may not mean "better," especially in terms of adding more hatchery or extraneous stocks to the system. Hatchery experts are looking at a variety of methods to enhance the quality of hatchery fish including better genetic management of the stocks. But if preservation of wild fish is the goal of the ESA, other more dramatic changes may be necessary.

One of these dramatic alternatives to Drawdown that the Corps is studying is the Upstream Collector. The objective is to screen virtually all of the outmigrating juvenile salmonids out of the Snake at a facility at Lewiston upstream of Lower Granite Dam and then transport the fish past dams in barges. NMFS has data from 1986 and 1989 adult returns that suggest fish in barges may yield better adult returns than fish in-river (Mathews, 1989). Others point to delays, straying and hyper-harvest of adults that were barged as juveniles (Olney et al, 1993). As one of its tasks to the Committee, Harza has been reviewing preliminary designs of upstream collectors as a possible alternative to Drawdown. Ideally, such a facility could sort fish by size and stock. A technical question that must be addressed if barging is in the salmon's future, is whether homing ability is impaired or delayed of adult salmon that have been barged as juveniles. The benefits of barging are controversial but they must be addressed within the context of Drawdown. Most reservoir Drawdowns will preclude barging of fish (or any product) down the river because navigation locks will be inoperable at lowered reservoir pools.

Harvest. On first blush, sufficiently large numbers of adult fish (about 2 million salmon and steelhead in 1991) return to support recreational and commercial harvest in the Columbia and Snake rivers. However weak stocks are mixed with strong stocks and wild fish with hatchery fish. As a result, if harvest includes 60% of all fish, on average, this will include 60% of stocks that are on the verge of extinction. Traditional fishing practice and economics generally govern harvest and there is a tragedy of the commons occurring. As many as 73% of certain troubled fall Chinook stocks may be harvested in-ocean even though they represent less than 1% of the ocean harvest. Harza is reviewing the various population models that are used to predict changing trends in adult returns. These include the Stochastic Life Cycle Model (Fisher, Lee and Hyman, 1992); the Empirical Life Cycle Model (Schaller, pers. comm.); and the Columbia River Salmon Passage Model (Fisher, 1992). Results of the Stochastic Life Cycle Model suggest that reducing adult mortality will lead to faster population recovery than similar improvements to any other life history stage of salmon. It is not the assignment of Harza to adjudge the merits or demerits of harvest. However, in order to adjudge the "biological efficacy" of Drawdown, all sources of mortality must be examined to know if Drawdown is likely to make a difference in the restoration of endangered or threatened stocks.

Engineering of Drawdown

Harza has reviewed the first draft System Configuration Study (Corps, 1992) which is a reconnaissance level report by Corps on the feasibility and cost of Drawdown and other alternatives. Harza found several significant cost saving measures and improvements to the schedule. These were presented to the Northwest Power Planning Council (Harza, December 4,

1992). Our primary recommendation was to reduce the number of Drawdown alternatives for feasibility study to three. These include (1) a fixed shallow (33-foot) Drawdown, (2) a deep (67-foot) variable pool Drawdown and (3) the Natural River option in which the entire reservoir would be evacuated through a newly constructed low level spillway 100 feet below current normal pool. For this last option, Harza recommended study of building the new outlet structure downstream of the existing dam obviating the need for expensive upstream coffer dams. Additionally, Harza recommended study of a downstream weir that would maintain normal tailwater and consideration of some type of top spill fish bypass that might be designed as a new side channel spillway. These concepts would retain a full range of Drawdown options, but allow more detailed feasibility level investigation as well as a more focused look at the biological efficacy of Drawdown.

Summary

There appear to be numerous sources of mortality of juvenile fish as they migrate. Delay of migration is one of them. If Drawdown is an appropriate strategy to enhance survival of outmigrating fish, it will be important to adjudge the relative merits of Drawdown in light of other existing sources of mortality. Additionally we must anticipate potential new sources of mortality in a changed system.

Based on our initial review of the biological data and expert opinions of a broad array of experts in the region, we have drawn the following tentative conclusions regarding the biological efficacy of Drawdown:

1. Although Drawdown could conceivably help migrating salmon that may be delayed by dams and reservoirs, we don't assume that Drawdown will be a panacea to all sources of mortality. In fact, it could exacerbate existing causes of mortality from predation, nitrogen gas saturation, adult migratory delay, lower fish guidance efficiencies and increased turbine mortality. All of these potential problems must be addressed prior to a final Drawdown design.

2. If mortality is extensive upstream of the reservoirs, Drawdown may do little to improve the survival of these fish. Most of these fish are of hatchery origin and their survival maybe less relevant to the ESA mandate to preserve wild fish.

3. If reservoir mortality is on the same order of magnitude as turbine mortality, say 10%, Drawdown, even if it is completely successful, may not be warranted in terms of the limited benefits that may accrue to the population.

4. If dams and turbines (not reservoirs) are contributing the bulk of juvenile mortality, Drawdown will not ameliorate these sources of mortality because the smolts will still need to navigate past these structures. Drawdown as currently conceived in the SCS Report could in fact exacerbate mortality because of anticipated decreases in fish guidance efficiency (FGE) and turbine efficiency at lowered reservoir pool levels.

5. If Drawdown is to have the greatest benefits, it must be designed such that juvenile fish are passed quickly around the dams and away

from the turbines thus avoiding delay from deadwater near the dam and avoiding mortality associated with turbines and predation in the tailrace. A system which utilizes top spill may have the best chance of providing these conditions. Perceived benefits for such a system can be drawn from the existing ice and trash sluiceway at Ice Harbor Dam and top spill at Wells Dam.

Harza engineers are currently reviewing the designs, costs and construction schedules of several Drawdown and non-Drawdown alternatives. These alternatives could save hundreds of millions of dollars and years of design and construction. Concepts include:

1. Use of the existing dam as an upstream cofferdam for construction of a new spillway outlet structure for the Natural River option,

2. design of a downstream weir that will allow existing fish bypass facilities to continue to operate, and

3. the design of juvenile bypass systems that utilize top spill to move fish around the dams and away from turbines.

Depending on progress made in the first half of 1993, we will report on additional new developments concerning the engineering feasibility of Drawdown and the biological advantage it may afford juvenile salmon.

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Considerations in Upstream Fish Passage

R.A. Alevras[1] and K.G. Whalen[2]

Abstract

The development of upstream fish passage on historically impounded Great Lakes rivers is an important consideration in hydro relicensing, and has both significant benefits and potentially adverse consequences. This paper focuses on the potentially adverse effects, which may have general applicability where upstream fish passage is a consideration in hydro licensing.

Great Lakes migratory fish may accumulate large body burdens of contaminants during the lake stage of their life cycle and may transfer these toxins to inland biota and the surrounding environment when their body tissue is consumed or decomposes after spawning or through egg deposition. The inland areas of some Great Lakes rivers support bald eagle populations and data suggest that the nesting success of eagles having access to Great Lakes fish is significantly lower than that of inland eagle populations.

Passage of Great Lakes fishes into headwater areas may adversely affect high quality resident fish populations by initiating competitive interactions between their offspring or by affecting the spawning activity of resident fish. Disturbance of these fisheries may thus have social and economic as well as environmental implications.

The lowermost dam on Great Lakes tributaries has served in many cases as an effective barrier to the spread of pernicious exotic species such as the white perch, alewife, gizzard shad, and sea lamprey; thus, with the develop-

[1]Program manager, Lawler, Matusky & Skelly Engineers, One Blue Hill Plaza Pearl River, New York 10965

[2]Project scientist, Lawler, Matusky & Skelly Engineers, One Blue Hill Plaza, Pearl River, New York 10965

ment of upstream fish passage comes the potential for the spread of these species.

Providing upstream fish passage on historically impounded Great Lakes rivers is an action that may result in the enhancement of some Great Lakes fish populations. However, that action may result in the introduction of toxics into uncontaminated inland areas, negative effects on resident fish populations, and the expansion of the range of pernicious exotic species. The decision on the implementation of upstream fish passage can be made only after careful consideration of many factors.

Introduction

Upstream fish passage could accommodate the interreach movements of resident river fishes as well as the spawning migrations of indigenous and introduced anadromous fishes resident in the Great Lakes. For the recreationally important chinook salmon, coho salmon, and steelhead trout, fish produced naturally downstream of existing dams contribute significantly to the adult lake populations. Thus, lake populations of Pacific salmonids might be enhanced and hatchery production reduced if these species had access to suitable upstream breeding areas. Angling opportunities would also expand if the range of these species was extended into upstream reaches. There are significant economic benefits associated with a range extension of these species.

The environment of the Great Lakes and the tributary rivers has changed substantially since the construction of dams in the early part of this century. The installation of upstream fish passage facilities should not be undertaken without consideration for the potentially adverse effects resulting from these changes.

Contaminant Transport: Pathways

An important consideration vis-à-vis the passage of fish from the Great Lakes through the hydro projects upstream to inland river segments would be the transport of contaminants into upstream areas currently free of substantial amounts of these contaminants. The primary contaminants of concern are the persistent PCB and DDT compounds (Walker 1976; DeVault et al. 1988). Because anadromous salmonids are exposed to these compounds in the lake stage of their life cycle and may accumulate large quantities of these toxins in their tissue (Rogers and Swain 1983; Baumann and Whittle 1988), passage of adults would be a conduit for the transport of PCBs and DDT into uncontaminated upstream areas.

Resident fish and wildlife of special concern may be exposed to and take up toxins bound in the tissue of contaminated adult salmonids by several pathways. Bald eagles, for example, forage primarily on rough fish (e.g., suckers, catfish), but they are also opportunistic scavengers that are influenced by food abundance (Frenzel and Anthony 1989). Bald eagles may use salmon as a food source during summer and fall runs, and dead and dying salmon during winter. Foraging on contaminated salmon during winter is of particular concern because it takes place just prior to the eagle nesting period. Mink and otter may feed directly on contaminated salmon. Because PCBs and DDT are difficult to eliminate from the body (Lamb et al. 1970 in Greichus et al. 1973), any wildlife feeding on contaminated fish may acquire high body burdens of these compounds.

Merna (1986) identified a direct pathway for toxins bound in the tissue of salmonids to transfer to the body tissue of resident fauna. Merna found that the eggs of coho and chinook salmon that had entered several Lake Michigan tributaries contained substantial levels of PCBs and DDT. Salmon eggs may constitute a large percentage, as much as 90%, of the diet of resident brook and brown trout in Great Lakes tributaries during the salmon spawning season (Johnson and Ringler 1979). Merna analyzed a number of fillets from resident trout and found that several contained PCBs in concentrations that exceeded the 2 mg/kg standard FDA action level and concluded that salmon eggs ingested by resident trout were the source of the contaminants present in the tissue of resident rainbow and brown trout. Thus, resident fauna can acquire contaminants transported into upstream areas by salmon, and biomagnification through the food chain (rather than bioaccumulation from sediments or bioconcentration from the water column) is an important pathway of exchange from contaminated salmon to resident fauna. This means that top-level carnivores, some of the inland area's most valued wildlife, would be at risk.

Contaminant Effects on Fish and Wildlife

Once introduced, PCBs and DDT compounds may affect fish and wildlife in a variety of ways. In brook trout and Atlantic salmon, for example, toxins such as DDT have been shown to affect the nervous system (lateral line function) (Anderson and Peterson 1969), thermal acclimation ability (changes in the upper and lower lethal temperature limits) (Peterson 1973), behavioral ontogeny (Saunders and Dill 1974), learning ability (Anderson and Prins 1970; Hatfield and Johansen 1972), predator/prey relationships (Hatfield and Anderson 1972), and reproduction (Macek 1968; Zitko and Saunders 1979). Adults of some salmonid species have innate ways of reducing DDT toxicity (Greer and Paim 1968; Cherrington et al. 1969). However, once toxins are assimilated into adult tissue, they may be passed on and become sequestered in the yolk tissue of eggs (Ankley et al. 1989)

and fry. The presence of contaminants in the yolk tissue is especially critical during the conversion from endogenous to exogenous feeding (Saunders and Dill 1974). As the yolk is metabolized, toxin concentrations increase and fry may be effectively poisoned.

In wildlife, PCBs and DDT have been documented as causing effects on both the individual and population level. Avian reproductive problems associated with PCBs and DDT include infertility, thin eggshells, embryo toxicity, structural embryonic deformities (Ludwig 1988) and damage to (McKinney et al. 1976; Gilbertson and Fox 1977) and inhibition of (Gilbertson 1975) enzymatic functions in the liver. PCBs in Great Lakes colonies of common tern, herring gull, and double-crested cormorant have caused varying degrees of effects, ranging from incomplete embryo development to incapacitating gross physical deformities, such as cross bill (Gilbertson 1975). On the population level, reproductive failure in double-crested cormorant has been positively correlated to levels of DDE (Vermeer and Peakall 1977). Organochlorine contamination has also been attributed to colony-size declines of herring gulls in Lake Ontario (Gilman et al. 1977). PCBs have been identified as the causative contaminant in the reproductive failure and increased mortality of individuals in mink populations (Aulerich 1973 and Aulerich and Ringer 1977 in Rohrer et al. 1982).

Recent studies have shown that contaminant concentrations (PCB and DDT compounds) in fish of the same species from the Great Lakes and inland lake populations were on the order of 10 to 100 times greater in Great Lakes populations. High concentrations of contaminants in fish in Great Lakes areas were directly related to reproductive failure and poor recruitment in double-crested cormorant colonies that have access to contaminated fish. In other parts of the country PCBs have also played a role in the decrease of populations of species of special concern, such as the common loon (Williams 1978).

Blood serum analysis of bald eagles by Bowerman et al. (in press) revealed that individuals from Great Lakes populations contained body burdens of both PCBs and DDT higher than those in individuals from interior populations. Bowerman et al. also determined that the nest productivity (measured as the number of nestlings per nest) of inland bald eagle populations that did not have access to contaminated fish was significantly greater than the nest productivity of bald eagles that consumed contaminated fish. The high level of contaminants in the Great Lakes compared to interior systems was identified as the primary cause of the difference in nest productivity between the two bald eagle populations. These data indicate that hydroelectric projects may act as literal barriers to the transport of contaminants into upstream areas by Great Lakes-borne species.

Fish passage is a long-term management objective; therefore, the long-term fate of contaminants is a concern. Although natural mechanisms that break down toxins exist (Furukawa and Matsumura 1976; Anderson 1980), once introduced into uncontaminated areas PCBs and DDT compounds may cycle through the ecosystem for many years. As a result of regulation and control, concentrations of toxins in the Great Lakes have been declining (DeVault et al. 1988). However, as Struger et al. (1985) have noted, lake sediments "represent a large reservoir of contaminants" and contaminants may therefore "continue to be available long after loading ceases." EPA, which has expended tremendous effort to curtail and restrict the introduction of toxins into the environment, noted (1990) that the dramatic restoration of important species such as the bald eagle was directly related to contaminant regulatory and control policies. Allowing contaminated fish to pass into uncontaminated areas could lead to the expansion of the range of toxic substances whose effects have been well documented (yet are not completely understood).

Interactions With Resident Fishes

Upstream fish passage will place adults and the juvenile offspring of exotic fishes in riverine habitats where they do not naturally occur. The introduction of exotic fishes has taken place many times in the Great Lakes, with significant adverse impacts to valuable resident sport and commercial species. Valuable resident trout fisheries in upper Great Lakes tributaries could be adversely affected by the introduction of migratory salmonids. Successful spawning of migratory salmonids may become a deleterious side effect of fish passage. Coho and chinook salmon spawn later than brook and brown trout and may superimpose their redds over those of resident trout (Avery 1974 in Johnson and Ringler 1979). The spawning activity of coho salmon has also been shown to significantly reduce the benthic invertebrate forage base in localized areas (Hildebrand 1971). Although this interaction has not been analyzed, adult resident trout and anadromous salmonids may compete for suitable holding areas in the stream. Finally, if migratory salmonids spawn successfully, their juveniles and resident trout juveniles may compete for available habitat.

Fausch and White (1986) believed that competition between juvenile Pacific salmonids and resident trout had the greatest potential to cause deleterious effects on resident trout. In coevolved species such as brook trout and Atlantic salmon, morphological (Webb 1988), physiological (Sosiak 1982), and behavioral (Kalleberg 1958) mechanisms serve to ameliorate competition. However, as noted by Fausch and White (1986) and Hearn (1987), competition is more likely if exotic salmonids are introduced into areas that contain resident fish that have habitat needs similar to their own. Juvenile

resident trout (brook and brown trout) and coho salmon in fact share a preference for pool habitat (Fausch and White 1986).

Several authors, noting that juvenile coho salmon and resident trout may compete for food or space in Great Lakes tributaries (Stauffer 1971, Avery 1974, and Taube 1975 in Johnson and Ringler 1979), have suggested that the presence of coho salmon affected resident trout populations. Controlled laboratory experiments by Fausch and White (1986) showed that coho salmon will in fact outcompete brook and brown trout juveniles when these species exist in sympatry. Coho salmon were more aggressive than brook and brown trout and forced resident trout species from preferred habitat into marginal positions. Fausch and White also observed that juvenile coho salmon fed effectively while defending space; brook trout on the other hand often stopped feeding after they had been displaced from preferred habitat. In natural systems coho salmon emerge earlier than brook and brown trout and are larger than juvenile resident trout during the growing season, which would enhance the salmon's ability to keep trout subordinate.

Based on the results of their study, Fausch and White (1986) intimated that fisheries managers should consider the effects that increases in coho salmon populations would have on resident brook and brown trout populations in Great Lakes tributaries. Because the effect of the passage of Pacific salmonids to upriver areas containing resident trout involves many forms of interaction (e.g., spawning activity, forage base, competition among juveniles), the specific ultimate effect of fish passage on resident trout populations is unpredictable and therefore may have dramatic unforeseen consequences.

Expansion of Pernicious Exotics

There is a potential to expand the range of a number of undesirable exotic species, including the white perch, alewife, gizzard shad, and sea lamprey, into inland waters through the development of fish passage facilities on Great Lakes tributaries. These species are among a long list of exotic fish species previously introduced to the Great Lakes. The white perch, a recent introduction that is spreading rapidly, has the potential to eliminate or reduce the abundance of more desirable game species. It has a history of rapid expansion in new habitats followed by adverse effects on existing fish communities. Alewife is a migratory marine species that invaded and became landlocked in the Great Lakes with a resultant significant adverse impact on the resident fish communities. Gizzard shad are present in many bays and at the mouths of Great Lakes tributaries. They have increased to nuisance levels in some areas and have had undesirable interactions with native resident species.

The sea lamprey is another nonindigenous species that has had a severe adverse impact on the Great Lakes ecosystem. The sea lamprey is ubiquitous in the region, inhabiting many Great Lakes tributary streams. Morman's (1979) review of the distribution of lampreys in Michigan's Lower Peninsula revealed how the lowermost mainstem dam on several Great Lakes tributaries has effectively restricted the sea lamprey invasion. Retarding the invasion of this species is highly significant; the Great Lakes International Joint Commission (IJC) estimated that $10 million has been spent annually on Great Lakes tributaries to control sea lamprey populations (IJC 1990).

While most fish passage facilities can be selective, no system is 100% effective at excluding undesirable species. Ryckman (1986) studied the effectiveness of fish ladders in the Grand River, Michigan. He found that in addition to migratory salmonids, suckers (*Moxostoma* spp. and *Catostomus* spp.), channel catfish, smallmouth bass, carp, and walleye used the ladders for upstream passage. The study shows that species typically thought of as nonmigratory will use a fish ladder. Upstream fish passage has the potential to facilitate the spread of undesirable exotic species that have proven to be environmentally destructive when introduced to new habitats. Efforts to minimize this risk would add significantly to the costs and potential risks associated with upstream fish passage, with no assurance that unintended introductions would not occur.

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Pool Dispersion Flow Net (PLDFLONT)

A Digital Computer Analysis and Simulation of Water Velocity Vectors (Magnitude, Direction, and Sense) in a Backwater Pool Resulting from Infusions from Upstream from a Control Structure into the Lower Pool

An Analysis and Program for Execution on an Electronic, Digital Computer

David N. Raffel[1]
Fellow of American Society of Mechanical Engineers

Issam M. Belmona[2]
Institute of Electrical and Electronic Engineers

An hydraulic simulation was used for determining the magnitudes and directions of the velocity vectors and the depths of water in the reach of the Ohio River downstream from Captain Anthony Meldahl Locks and Dam. The simulation was made for cases of various total Ohio River flows both without and with the City of Hamilton, Ohio's proposed hydroelectric project.

While the analysis may not be perfect, it improves over anything previously available. Further improvement could be made, accounting for wave reflections from the boundaries. However, computer running time would then substantially increase. Considering the imprecisions in the data necessarily incorporated into the model, and considering the imprecise knowledge of fish conduct and responses to changes in habitat, the authors believe the mathematical model described in this presentation serves a useful purpose by improving over what has been done in the past.

[1] President, Protrans Consultants, 4150 Rio Bravo Street, Suite 242, El Paso, Texas 79902

[2] Associate, J.S. Sawvel and Associates, 100 East Main Cross Street, Suite 300, Findlay, Ohio 45840

During physical hydraulic model studies conducted for the Belleville Hydroelectric Project on the Ohio River in 1993, we will have the opportunity for comparing the mathematically modelled velocity vectors with those measured on the physical model.

Most IFIM (Instream Flow Incremental Methodology) studies are made using a standard hydraulic model. That model assumes that the water released to downstream from the site is introduced into a free-flowing stream. The water surface elevations in the free-flowing stream are the consequence of the local geometry and the hydraulic characteristics, with no influence from downstream controls. At the Ohio River sites being considered for installing hydroelectric plants, that is not the case.

At Meldahl, for example, the water surface elevation downstream from the dam is influenced primarily by the quantity of water flowing in the river and by the backwater effect of the pool level maintained at Markland Locks and Dam, about 95 miles downstream. At zero river flow, the water level just downstream from Meldahl Dam will be identical to the pool water level maintained at Markland Dam. For all of the total river flows of interest for the IFIM study, the water level of the pool at Markland Dam is maintained at elevation 455.0 feet. As the river flow increases, the water surface in the reach of river between Markland and Meldahl tips, higher at Meldahl, so the slope can overcome the friction resisting the flow in the 95 miles of river channel. This situation is not covered by the standard IFIM hydraulic simulation models. A new hydraulic simulation was developed, which accurately analyzes and predicts the hydraulic conditions downstream from Meldahl dam, with and without the proposed hydroelectric project.

This is a description of the PROTRANS/SAWVEL proprietary program, PLDFLONT (Pool Dispersion Flow Net), for electronic, digital computer simulation of dispersion of water introduced from single or multiple infusion places into a backwater pool in a natural channel. The analysis determines the velocities and directions of flows, and the depths of water. It was used for the City of Hamilton, Ohio's IFIM study of habitat for marine organisms downstream from Meldahl Dam.

The input includes a numerical description of the channel bottom and side boundaries topography. The topography was developed from field soundings taken by Woolpert Consultants in the Ohio River on September 23 and 24, 1991, while the total Ohio River flow at Meldahl was the lowest of the year, about 15,000 cubic feet per second. A portion of the side boundary topography above the water surface was taken from existing Corps of Engineers maps. The field soundings were taken at twelve points on each of five transects parallel to the axis of the dam. The transects were at 300 feet, 500 feet, 1000 feet, 1500 feet, and 2000 feet downstream

from the axis of the dam. In addition to the soundings to know the depth from water surface to bottom, the velocity of flow, and the dissolved oxygen content were measured. Samples also were taken by coring at the sounding locations, to know the character of the river bottom, that is the substrate, at all locations where the bottom was not determined to be solid rock or large boulders.

Knowing the tailwater elevation, which is a unique function of the total river flow when the Markland pool is maintained at a constant elevation, the elevation above Ohio River datum of the bottom was then determined from the tailwater surface elevation rating curve for the site less the depth as sounded.

Those bottom elevations were then used for developing a topographic map of the river channel, by the same methods as are used to develop a topographic map from any land survey. Once the topographic map of the river bottom was available, a uniform X-Y grid was overlaid on the topography, and bottom elevations at each grid node were interpolated from the topography. Thus, the data from twelve soundings at each of five transects was converted to a grid of 20 by 22 cells each 100 feet by 100 feet.

Input to the PLDFLONT program includes the numerical description of the channel bottom and side boundaries topography. Input also includes selection of Manning's "n" friction characteristics of the channel, a tailwater rating of elevation of the water surface versus total Ohio River flow at the site, and the locations and magnitudes of the sources of infusions of water into the Markland backwater pool at Captain Anthony Meldahl Locks and Dam, and from the proposed hydroelectric project.

Output from the analysis includes velocity vectors (magnitude, direction, and sense) within the Cartesian coordinate X-Y grid, and the depth of water above the river bottom at any point within the grid.

The PLDFLONT program is sufficiently general that it can be applied to any backwater pool, matching velocities and channel topography measured in the field. Velocities computed with the program were compared with velocities measured in the field at Meldahl as the test, or calibration, case. Velocities were very low on the measuring days, because the year's extreme lowest flows existed on those days. The test case computer results adequately matched the velocities measured in the river. The computed results using the PLDFLONT program are probably more accurate than what could be measured with the velocity meters operating at the low end of their useful range.

Basis for Computing Velocities and Directions of Flow

The best obtainable from any physical or mathematical model is a first approximation of the prototype situation. It is known from other work that the friction characteristics of the various flow paths can be modeled. Momentum is conserved. That is, the mass times the velocity at time one equals the mass times velocity at time two. The flows and their physical consequences can be superposed, one atop the other, and resolved into a resultant flow. All studies of hydraulics of flow in open channels are based on those concepts. The concepts have been proven valid by field measurements of real conditions.

The earliest significant investigations were carried out by de Chezy (de Chezy, A., Mem. de Classe de Sciences de l'Inst. de Paris, 1813/15). De Chezy found that:

$$V=C\sqrt{RS}$$

Where	V	=	Velocity of flow, feet per second
	R	=	Hydraulic radius, feet = Area of the water in the cross-section, sq. ft/wetted perimeter, ft.
	S	=	Friction slope, feet per foot length
	C	=	Coefficient in de Chezy's formula.

For any given section, the de Chezy C can be found by substituting Manning's Equation (Manning, Trans. Inst. Civil Eng.,Ireland, 1890):

$$V=(1.486/n)R^{1/6}\sqrt{RS}$$

When the channel section is much deeper than it is wide, we can assume R is nearly the same as the depth.

Assume water is infused into the pool from one point. The water moves away from the point of infusion in circles of increasing radii. For a portion of the circle, an arc one foot in length, the amount of water flowing through a portion of the arc with depth = d, Q, is

$$Q=Vd$$

Further, assume that the resistance to flow, the friction slope (not the water surface slope, since the water surface elevation is controlled by the backwater from Markland Dam, the downstream control) is S. At any point along the same arc, the friction slope is

$$S=\frac{V^2}{C^2R}=\frac{V_A^2}{C_A^2R_A},etc.$$

R is known
S is assumed, as a first trial

$$C=(1.486/n)R^{1/6};V=(1.486/n)R^{1/6}\sqrt{RS};V=(1.486/n)R^{2/3}S^{1/2}$$

For Meldahl, Manning's friction factor, "n", was assumed to be 0.020. That is appropriate for a relatively smooth, natural channel, free from growths, with little curvature. It is also appropriate for very large canals (such as the man-made tailrace) in good condition.

Refer to Bresse (Bresse, J.A.C., Mechanique Applique, V.2, Paris, Mallet-Bachelier, 1880) and Woodward and Posey (Woodward, S.M. & Posey, C.J., Hydraulics of Steady Flow in Open Channels, New York, Wiley, 1941). Computations based on the hydraulic radius of the entire cross section will be in error when part of the flow is in a deep, natural channel and part is in shallower sections, as depicted in **Figure 1**, below:

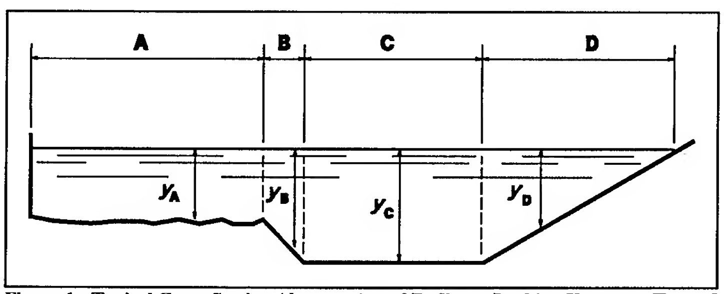

Figure 1: Typical Cross-Section Along an Arc of Radius x, Looking Upstream Toward Meldahl Dam

For that reason, flow in A, B, C, and D subsections of the total flow cross-section should be computed separately. The areas and wetted perimeters of each subsection end at an imaginary vertical line separating one portion from another.

The total flow Q_t through an arc of four sections can be expressed as

$$Q_t=\sum_A^D Q_n=\sum_A^D AV$$

The area of a portion of the arc through which the water is flowing, A, equals the average depth for that portion, d, times the length of arc, l. Then, the discharge through that portion of the arc is

$$Q=AV=dlV$$

For a single infusion, where Q is the total flow being infused into the lower pool,

$$Q=\sum_{A}^{D} dlV$$

$$Q=\sum_{A}^{D} dl(1.486/n)R^{2/3}S^{1/2}$$

As a simplified case, if the depth were uniformly 15 feet and the total length of the arc 2000 feet, with S=0.000005 (the first trial) and "n"=0.020,

Q through the arc = 30,315 cubic feet per second.

If the known amount of water infused was different than 30,315 cubic feet per second, adjust the value of S and try again.

The R and the average d may vary for each portion, A through D, of the arc. If we assume that the friction slope for each portion of the arc through which water flows is identical, then the elevation of the water surface will be the same anywhere along an arc of a given radius. That is a good first approximation of what happens when water is infused below the elevation of the surface into a backwater pool. The physical "bloom" of water, where hydraulic turbine discharge leaves a draft tube and enters the tailwater beneath its surface, can be seen from the powerhouse or the shore. The "bloom" of slightly raised surface elevation disappears within a few feet.

Assuming the same friction slope for each portion of the arc weights the flows through each portion in accordance with the geometry. By this means, we account for the tendency of the water to distribute itself according to paths of equal resistance to flow.

Calculating the Velocities and Directions of Flow

No attempt is made to distinguish between water velocities near the free surface, water velocities at mid depths, and water velocities near bottom or side boundaries. With the bottom topography known, Manning's "n" known

or assumed, and the quantity of water introduced into the lower pool through one infusion point known, the lower pool water surface elevation (for determining the depth, d) from the tailwater rating, the velocity, V, can be determined for any part of the arc of the circle of radius X_1.

Assume a trial S for portion A of arc X_1, and compute V for portion A. Assume the same trial S and compute V for portion B. Assume the same trial S and compute V for portion C. Assume the same trial S and compute V for portion D. The discharge, Q = Area times V. Then check. Does the sum of the discharges for each of the portions of the arc equal the total discharge through the arc? If it does, store the velocities and go on to the arc of next greater radius. If the total discharge does not check, adjust the trial S, and recompute the velocities.

For example, assume portion A of arc X_1 is 250 feet long, and the depth, d, equals 15 feet. The area is 15 x 250 = 3,750 square feet. The hydraulic radius, R, = Area divided by wetted perimeter = 3,750/280 feet. Assume the trial friction slope = 0.000005 feet per foot. Then, velocity through that portion of the arc = 1.643 feet per second, and the discharge through that portion of the arc = AV = 3,750 times 1.643 = 6,162 cubic feet per second.

Suppose the sum of discharges, similarly calculated, through portions A, B, C, and D of the arc = 27,000 cubic feet per second. But suppose the known total quantity of water infused and passing through that arc = 24,000 cubic feet per second. Then adjust the trial friction slope, using 24/27 times 0.000005 = 0.00000444 feet per foot and try again. A reasonable matching of the total flow can be obtained within a few trials. Store the final velocities through each portion, since they will be needed later for the vector resolutions.

Proceed by radial increments from the point of infusion. Assume all of the individual arcs of circles radiating from the point of infusion discharge into the tailrace or river downstream from Meldahl Dam. Assume that all the arcs terminate at the upstream to downstream location of the place at which water is assumed to enter the lower pool.

Water velocities will be relatively low, in the range from zero at no flow to about six feet per second during a great flood, in most of the potential habitat volume immediately downstream from Meldahl Dam. The reach of Ohio River from the axis of the dam to the farthest downstream location studied, and from the river side of the lock wall to the Kentucky shore, can be divided into a Cartesian X-Y grid. A velocity vector falling within a grid square describes the velocity anywhere within the square, if the square is sufficiently small. The same vector could be assumed in adjacent squares which do not contain the loci of a new vector. Where more than one vector

could be translated to a square from different adjacent squares, the vectors can be resolved to denote the square with the resolved velocity vector.

Velocities along radial paths, and within a Cartesian grid, have been demonstrated for infusion of water from a single source. There usually will be a second, a third, or more infusions. The locations of each infusion will be at different places. For example, the first may be the powerhouse, the second may be water released beneath Gate No. 12, the third may be water released beneath Gate No. 4, etc. Each infusion of water will create its own set of velocities along paths radiating from the new origin. Each of the sets of velocities can be computed independently.

The Principle of Superposition applies. Finding the resultant sets of velocity directions and magnitudes is a matter of adding the vectors and resolving them for each grid square. For the Meldahl IFIM study, up to 20 times 22 = 440 habitat squares were used. Some of the vectors were the resultants of previous resolutions.

If there were no leakage beneath the submergible gates at Meldahl Dam, and no flow though the locks (as a very simple illustration), the situation with the proposed hydroelectric project might look like **Figure 2** below:

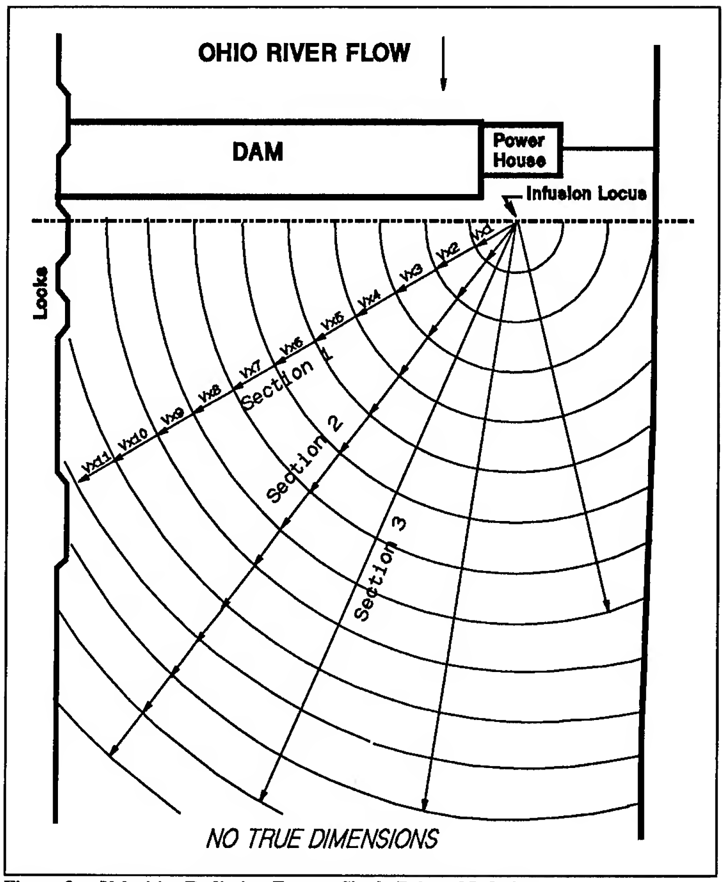

Figure 2: Velocities Radiating From a Single Point of Infusion

The velocities at a given radius will be different from one another at different angles, because of different local channel geometry.

At an identical total river flow, again assuming no leakage and no lockage (for simplification only, since this will never be the situation), the velocities might be as shown in **Figure 3**, below. In **Figure 3**, two infusion loci are shown, and a Cartesian X-Y grid is laid over the area of study.

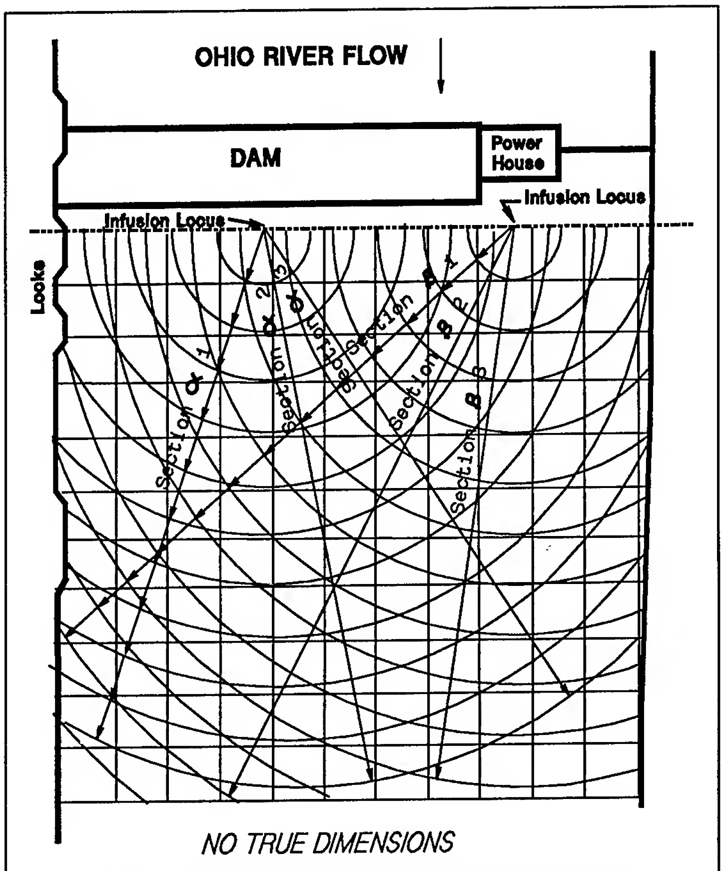

Figure 3: Velocity Vectors Radiating From Two Point Sources of Infusion

Velocity vectors from each of the infusion sources in each grid square are then resolved into a single vector representative of the square.

For Meldahl cases without the proposed power project, at total Ohio River flows of 10,000, 20,000, 30,000, 40,000, 50,000, 56,360, 80,000, 100,000, 200,000, and 250,000 cubic feet per second were studied. An additional condition, for 15,000 cubic feet per second, was also studied as the test case. The 56,360 cubic feet per second matches the total river flow when discharge through the proposed power project is maximum. The cases with 250,000 cubic feet per second flow occur at a time when the turbines of the

power project would be shut down because the head would be too low for safe operation. The infusion loci and the quantities of water infused matched the existing "Gate Operating Schedule for Improved Reaeration" used by the Corps of Engineers. The same flows were studied with the proposed powerhouse operated for maximum energy production. The results were compared from the viewpoint of effect on the habitat for marine organisms.

Resolving the Direction and Sense of the Velocity Vectors

There can be several locations where water is discharged from upstream from the control structure and infused into the lower pool. Each point of infusion has an X and a Y coordinate in a Cartesian grid. For example, power plant discharge may be at coordinates X_1 and Y_1. Flow under spillway gates may be infused at coordinates (X_2,Y_2), (X_2,Y_3), etc.

Assume we are calculating the velocity at point (grid node) Z(XY). After a few iterations, the magnitude of the velocities V_k (where k=1,2,3,...n) at point Z will be calculated as already described. The direction of the vector is still unknown, and must be calculated.

$$|V_k| = known$$

$$UnitVector{:}\, U_k = \frac{(X-X_k)}{\sqrt{(X-X_k)^2+(Y-Y_k)^2}}\,i + \frac{(Y-Y_k)}{\sqrt{(X-X_k)^2+(Y-Y_k)^2}}\,j$$

$$V_Z = \sum_{k=1}^{n} |V_k| \cdot U$$

Using the PLDFLONT Program

As applied to the pool below Captain Anthony Meldahl Locks and Dam on the Ohio River, there can be up to 15 locations where water is discharged from the upper pool and infused into the lower pool:

	1	location for combined discharge from both locks.
up to	12	locations for discharge beneath, or leakage beneath spillway gates.
either	1	location for combined discharge from the turbines, or
up to	2	locations at the south (Kentucky) end of the dam, for overflow over the fixed concrete weir or over the permanent cellular cofferdam and crib.

The turbines at the proposed hydroelectric plant would normally be shut down, with wicket gates closed, at times when the upper pool level is higher than the elevation 487 feet crest of the concrete gravity overflow weir or the proposed permanent cellular cofferdam and crib extension and seepage cutoff south from the powerhouse.

The depth, d, at any place in the grid equals the elevation from the tailwater rating at the total flow (i.e. the sum of flows from all infusion places) less the elevation of the channel bottom at that place in the grid. The grid dimensions and intervals will depend on the amount and the quality of information available, on the overall dimensions of the area being studied, and on the abruptness with which the topography changes. For Meldahl, a grid interval of 100 feet was selected. The grid extended to 2,000 feet downstream from the axis of the dam. Further downstream, water released through the powerhouse will have already mixed with water released from other places, and there will be no change from the existing channel boundary geometry.

There may be situations in the general program in which Manning's "n" varies at different places within the Cartesian grid. In such cases, the program input must also include a two-dimensional table of Manning's "n" versus coordinate position. If the Manning's "n" does not vary, the program can use a single input "n" and bypass the table lookup.

The PLDFLONT program can be used as an interactive program, or can be operated from tables of input cases data. In the latter operation, the instructions for the printout headings must be listed for all the cases. Interaction includes requests for these inputs from the keyboard:

Is the case being run with the power plant operating?

Is the operation to be interactive, or is the program to read from a file **INPUT.DAT**?

What is the total flow infused into the lower pool from all places?

How many infusion places are appropriate for the run?

What is the increment by which the radius of the arc is increased?

Enter a one-line description of the case being run.

Is a single Manning's "n" being used? If so, enter the value of "n."

Enter the amount of water infused into the lower pool, and the X-coordinate, for each place where water is released from the upper to lower pool.

As used for Meldahl, the X-coordinates for the possible places of infusion are the following:

	feet
Locks	2,200
Gate 1	2,135
Gate 2	2,020
Gate 3	1,905
Gate 4	1,790
Gate 5	1,675
Gate 6	1,560
Gate 7	1,445
Gate 8	1,330
Gate 9	1,215
Gate 10	1,100
Gate 11	985
Gate 12	870
Weir	758.5
Powerhouse	556
Cofferdam/Cutoff	224

It should be noted that the width of channel available for flow, and the depth both increase on the left (south) bank of the Ohio River, after excavation of the tailrace channel for the proposed hydroelectric project. The velocities will be changed, depending on the locations of the infusions of water, the geometry of the channel boundaries, and the amounts of water being infused at each location.

As used for the Meldahl Project, the geometry of the channels with the proposed hydroelectric plant is included in a file **BOTTOM1.DAT**. Without the proposed power plant, the geometry of the channel is included in a file **BOTTOM2.DAT**. Additional input files include the tailwater rating, **TAIL.DAT**, and the X and Y coordinates for the south floor of the excavated tailrace (the corner where the excavation slopes up until it meets the existing ground surface), a file of pairs of X and Y called **XCOORD.DAT.** The vector resolutions are handled in subroutines called up from the main program.

For example, for Meldahl, the grid increment was 100 feet. The program reads the width of the grid, that is the width of the river and overbanks along the axis of the dam, 2200 feet for the flows being studied, with the origin near the railroad tracks on the Kentucky side of the river, and extending to the river side of the lock wall. The program also reads the length downstream being studied. For Meldahl, the length for the IFIM study was 2000 feet, beginning at the axis of the dam, and ending upstream from where Big Snag Creek Sand Bar begins, since the tailrace channel from the powerhouse has already returned to the main river channel within that length.

The maximum arc radius will be greater than the maximum "X" or "Y" coordinate, since the radius can be a diagonal. The program calculates the maximum arc length that can fall inside the grid. If different Manning's "n" is being used for various places within the grid, Manning's "n" is read from an input file, **MANNING.DAT**.

The second input file used is called **TAIL.DAT**. The program opens that file and finds the water level at the upstream end of the Markland pool corresponding to the tailwater level for the total Ohio River flow at the site. If the flow is between two numbers in the data file, the value of the tailwater level (**TAIL**) is interpolated.

Cases with the Power Plant in Place

The third input file is called **XCOORD.DAT**. This file has a Y coordinate and a corresponding X coordinate for the corner of the floor of the excavated tailrace channel which is closest to the railroad tracks. As used for Meldahl, the channel bottom is divided into four portions of each arc. The X coordinate of that corner of the tailrace floor is needed for calculating the X coordinate of the Kentucky side top of the slope from the floor to the existing ground, by the following formula:

$$X_5 = X_4 - [(TAIL - 414.0) \cdot 4.5]$$

The slope of the bank side channel excavation is assumed, for this case, to be 4.5 horizontal to 1.0 vertical. The bottom of the channel, line 3 to 4, is assumed to be at top of rock, about elevation 414.0.

For Meldahl, the river side of the tailrace channel, corner 3, has a bottom elevation 414.0 at X coordinate 760 feet. Then there is an upward slope to the existing river bottom. The X coordinate of the top of that slope, point 2, is calculated as

$$X_2 = 760 - [(Channel bottom elevation - 414.0) \cdot 2.5]$$

That is, the riprapped slope from the bottom of the tailrace to the existing river channel bottom, line 3 to 2, has a slope assumed to be excavated at 2.5 horizontal to 1.0 vertical.

The program then opens another input file, **BOTTOM1.DAT,** and reads the bottom elevation of the river (the bottom topography as developed from the field measurements). For Meldahl, data in this file are in X and Y coordinates starting 900 feet away from the X origin and one hundred feet away from the Y origin.

The elevations of the river channel bottom from X = 100 to X = 900 are calculated, for Meldahl, based on the following formulae:

$$Y=AX+B, for X_5 \leq X < X_4$$

$$A=\frac{TAIL-414}{X_5-X_4}$$

$$B=TAIL-AX_5$$

$$Y=414, for X_4 \leq X < X_3; Y=40X+110, for X_3 \leq X < X_2$$

The program then calls a subroutine, **GEOMTA**, which performs the following tasks:

First, the average bottom elevation in feet is computed for each portion of each arc. This is done by taking many readings from the input file, **BOTTOM1.DAT**, which fall on that portion of the arc, and taking the average. For Meldahl, four bottom elevations were averaged for each portion of each arc.

Second, beginning and ending X and Y coordinates are computed for each portion of each arc. The lengths of the arc portions are then computed. The subroutine then returns to the main program the bottom elevations of each portion of each arc (**SECTD**) and the length of each portion of each arc (**SECTW**) for use in the main program.

The velocity is computed based on the following formulae:

$$S=0.000005$$

$$A=SECTD\times(Tail-SECTD)$$

$$R=\frac{A}{SECTW+2\times(TAIL-SECTD)}$$

$$C=\frac{1.486}{n}\times R^{\frac{1}{6}}$$

$$V=C\sqrt{RS}$$

After the velocities are computed, the total flow through all the sections of the same arc is computed. The program adjusts the value of S, the friction slope, to a value which will bring the total flow to the known flow from the infusion point. This is repeated until we have the values of magnitudes of velocity for each portion (section) of each arc consequent from each source of infusion of water into the lower pool.

The main program then calls another subroutine, **VELOCA**. This subroutine computes the velocity in magnitude and angle direction at each point in the Cartesian grid. The magnitude at each node is computed from each infusion source by locating a point in the grid between two arcs of a circle whose center is at the source of the infusion. This is an interpolating computation. The closer the grid node intersection is to the arc, the closer is the velocity to the velocity of flow through the arc.

A single unit vector is then computed by resolving the individual vectors from each of the sources of infusion into a composite vector representing the consequences of flow from all of the sources of infusion of water into the lower pool. Those resultant vectors at each node in the grid are returned to the main program for printing to an output file, named **OUTPUT.DAT**.

OUTPUT.DAT includes the X coordinate and the Y coordinate of each point in the grid, the magnitude of the velocity in feet per second, the angle of the flow in degrees (90 degrees is perpendicular to the axis of the dam, and facing downstream), and the depth of water at that point. The last thing the program will do is call time and compute the elapsed time since starting the case run. The elapsed time is printed at the end of the program run.

Cases Without the Power Plant

Without the power plant, bottom elevations are taken from the file **BOTTOM2.DAT**. All the data for the existing bottom and side elevations from X = 100 to X = 2200, and from Y = 100 to Y = 2000, in increments of 100 feet, are in **BOTTOM2.DAT**. At this point, all the data needed for executing the program are read and stored in the variables and arrays.

A subroutine called **GEOMTB** is then called. This subroutine performs computations similar to those of **GEOMTA**, taking into account that there is no power plant and no tailrace channel. The arc sections are of equal length, at 45 degree intervals from origins at each infusion point. Thus, there are four equal portions on each arc.

At present, the PLDFLONT is programmed for use in either an OS/2 or a DOSv5 operating environment, with 2 Mb minimum RAM. The output printing routines require Lotus 123v3.1. Earlier versions of DOS or Lotus will not handle the work. A single run takes 30 to 45 minutes, without a math coprocessor, using a 386-based PC at 16 MHz or faster.

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STANDARDIZING INSTREAM FLOW REQUIREMENTS AT HYDROPOWER PROJECTS IN THE CASCADE MOUNTAINS, WASHINGTON[1]

Ian M. Smith, P.E., M. ASCE[2], and Michael J. Sale[3]

Abstract

Instream flow requirements are common mitigation measures instituted in the bypassed reaches of hydroelectric diversion projects. Currently, there are two extremes among the ways to determine instream flow requirements: generic standard-setting methods and detailed, habitat-based, impact assessment methods such as the Instream Flow Incremental Methodology (IFIM). Data from streams in Washington state show a consistent pattern in the instream flow requirements resulting from the IFIM. This pattern can be used to refine the simpler standard-setting approaches and thereby provide better estimates of flow needs during early stages of project design.

Introduction

Many small, high-gradient, headwater streams with tremendous potential for hydroelectric development are located in the Cascade Mountains

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[2] Postgraduate fellow with the Oak Ridge Institute for Science and Education at ORNL, P.O. Box 2008, Oak Ridge, Tennessee 37831-6036 (615/574-1755).

[3] Hydrosystems Group Leader, Environmental Sciences Division, ORNL, P.O. Box 2008, Oak Ridge, Tennessee 37831-6036 (615/574-7305).

of Washington state. The Skagit and Nooksack river basins are typical of the Cascade region, and numerous small hydroelectric developments have been proposed in both areas, some of which are shown in **Figure 1**. The proposed hydropower developments are high-head projects with small diversion structures routing water around a reach of the affected stream (bypassed reach) to a powerhouse some distance downstream and then back to the stream below the powerhouse. The primary environmental effects of project operations are upon aquatic habitat and related natural resources in the bypassed reach of the stream.

An instream flow requirement (IFR) is routinely established in the bypassed reach of these diversion projects to protect aquatic habitat and fish resources. For the purposes of this analysis and discussion, an IFR is defined as the streamflow that must be maintained at all times within the bypassed reach, unless the naturally occurring flow at the diversion point is less than the IFR. Both the economic viability and the environmental impact of a project are strongly dependent upon the IFR. IFRs are determined by a number of different methods ranging from simplistic standard-setting approaches that do not require extensive field data to incremental habitat evaluation and impact assessment methods (Lamb and Doerksen 1985). One of the most common standard-setting methods is the Montana, or Tennant, method that is nothing more than a percentage of the mean annual

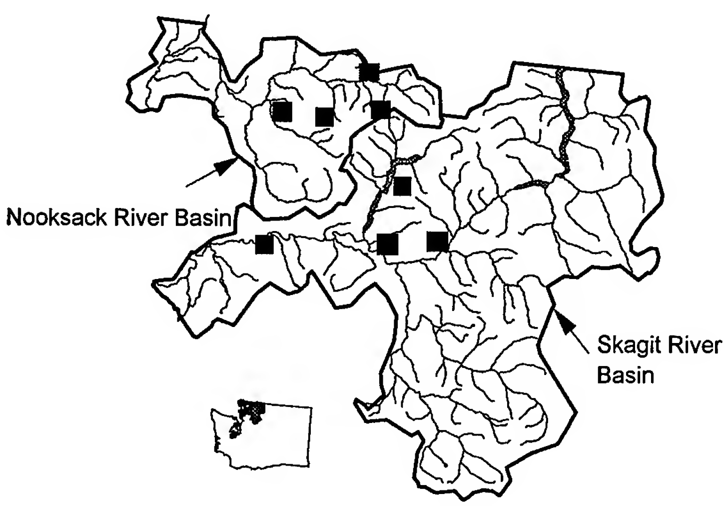

Figure 1. Map of Skagit and Nooksack River basins, with instream flow study sites denoted by squares.

flow at a site (Tennant 1976). The Instream Flow Incremental Methodology (IFIM), and its components such as the Physical Habitat Simulation (PHABSIM) system, is the most common habitat evaluation, or project impact assessment method (Bovee 1982, Stalnaker 1993).

The existing methods for determining IFRs can be arranged in a hierarchy according to their expense and complexity. Standard-setting methods, such as setting the IFR to a fixed percentage of average stream flow, are inexpensive and relatively easy to implement but are not useful for comparing alternatives in environmental impact studies. Impact assessment or habitat assessment approaches can be tremendously complex, with expenses to match. For hydroelectric projects with capacities between 1 and 10 megawatts, the average cost of applying a project impact assessment approach is over $46,000; the average cost escalates to over $230,000 for projects with generating capacities of between 10 and 50 megawatts (Sale, et al 1991). However, project impact assessment approaches allow comparison of different project alternatives (Trihey and Stalnaker 1985, Stalnaker 1993).

In practice, IFR assessment methods have evolved into three categories based on when they are used in the course of project development (Trihey and Stalnaker 1985): reconnaissance studies, feasibility studies, and operational design studies. The degree of site-specific field data, and therefore cost and complexity of application, increases respectively. Standard-setting methods are most often applied in the first two stages; incremental methods are not usually applied until the operational design phase. Incremental, project impact assessment approaches provide the most benefit if applied in the feasibility study phase, earlier than has been usual. Earlier application of these incremental methods would allow earlier identification of the most economically and environmentally sound alternatives, or dismissal of an infeasible project. Despite their advantages, incremental methods are seldom applied at this point in the process due to their costs in time and money. They are most often applied during or even after operational design.

The purpose of this paper is to demonstrate how information from the more detailed habitat evaluation IFR methods can be transferred to unstudied sites and to earlier stages of development. The resulting procedure will allow reconnaissance or feasibility studies to benefit from results of the project impact assessment and IFR negotiation process undertaken for very similar ecosystems, while avoiding the higher costs of a full-blown IFIM study until later in the project development process. This procedure can also be used to improve standard-setting methods.

Approach

Our initial approach for this study was to look for significant correlations between: (1) independent watershed and hydrologic characteristics of streams, and (2) the IFR values that resulted from full application of the IFIM. The IFR and watershed data were collected or derived from license applications and supporting documentation filed for eight proposed or existing hydroelectric projects in the Skagit and Nooksack River basins (**Figure 1**). The IFIM studies supporting the IFRs are the end result of extensive consultations between the applicant and state resource agencies. The IFIM results characterize the relationships between stream flow and usable habitat area for the salmonid species and life stages of interest. Negotiations are then conducted to determine the instream flow requirements that best meet the resource agencies' habitat objectives. Both sides refer to the IFIM-based habitat-flow relationships for support of their IFR positions. Once an agreement is reached, the results are recorded in an agreement report.

Table 1 lists the site data, annual average IFR values, and habitat objectives of the IFIM study and negotiations for each data point used in the analysis. Annual average IFR was the dependent variable used in the

Table 1. Site data and instream flow requirement (IFR) values used in analysis (project locations are shown in Figure 1).

Stream	Drainage area above diversion (km^2)	Average Gradient (%)	Average Annual Flow (m^3/s)	Average Annual IFR (m^3/s)	IFIM Habitat Objective[a]
Nooksack Falls	178.6	9.7	14.8	0.93	POC
Wells Cr.	56.7	7.5	4.9	0.85	POC
Glacier Cr.	35	5.3	3.1	0.74	NNL
Anderson Cr.	8	16.5	0.7	0.36	NNL
Rocky Cr.	15.5	15.7	1.9	0.40	NNL
Jackman Cr.	24.3	4.6	2.8	0.41	NNL
Canyon Lake Cr.	11.3	13.8	0.6	0.17	MHL
Olson Cr.	11.1	20.0	0.5	0.10	NNL

[a] Definition of IFIM habitat objectives: POC = Peak Of Habitat Curve, NNL = No Net Loss, and MHL = Minimize Habitat Losses.

analysis. Many IFRs varied seasonally or monthly. To obtain a single to work with, we calculated an average IFR value for each stream by multiplying each seasonal or monthly IFR by its duration in days, summing these products, and then dividing by 365 days.

Our analysis is based upon several assumptions. First, the available data from eight IFIM studies are representative of the whole population of Cascade mountain streams. Secondly, application of the IFIM is appropriate for these streams. Thirdly, average annual IFRs are a valid distillation of IFIM results. Finally, similarities in fish species and life cycles among streams create the need for similar seasonality in IFRs.

We examined the correlation between annual average IFR and stream characteristics using linear, least-squares regression applied to the eight data points. In addition to the regression analysis, the Tennant Method was applied to the eight study streams. We calculated instream flow requirements in accordance with Tennant (1976) to provide two levels of habitat maintenance: "poor" (IFR = 10% of annual average flow) and "excellent" (IFR = 30 to 50% of annual average flow) in Tennant's terminology. These IFRs provided a basis for comparison with the IFIM-derived data points and the regression results.

Results

Five of the eight IFIM-based IFRs were non-constant, demonstrating strong and consistent seasonality. The seasonality is very similar to the seasonal variation of streamflows for the subject streams. In conjunction with the single IFR predicted by the regression, we used the IFR variability to incorporate seasonal variation into the regression result. There is a high-IFR season (June-September), a low-IFR season (December-March), and two transition periods (October-November, and April-May). From observation of the data, the high IFR plateau corresponds to approximately 1.5 times the annual average IFR, while the low IFR regime equates to 0.5 times the annual average. The two transition periods are approximated by the annual average IFR. The seasonal IFRs, weighted by the durations of their corresponding seasons, maintain the annual average IFR.

The first and simplest relationship we looked at was between the negotiated IFR and average annual flow (QAA). Regression analysis resulted in the following equation for annual average IFR:

$$IFR = 0.316 + 0.577 \log_{10} (QAA) \quad (1)$$

where IFR = average annual instream flow requirement (m^3/s); and
QAA = average annual flow in the bypassed reach (m^3/s).

The correlation coefficient (r^2) for the regression is 0.863 with F = 37.8 (Prob. > F less than 0.05).

Two other independent variables accounted for much of the observed variability in the annual average IFRs (**Figure 2**): $\log_{10}$(drainage area above diversion point), and average streambed gradient of the bypassed reach. Regression analysis resulted in the following equation for average annual IFR as a function of these two variables:

$$IFR = -0.119 - 0.011\ G + 0.534\ \log_{10}(A), \qquad (2)$$

where IFR = average annual instream flow requirement (m^3/s);
G = average gradient of bypassed reach (%);
A = watershed area above diversion point (km^2).

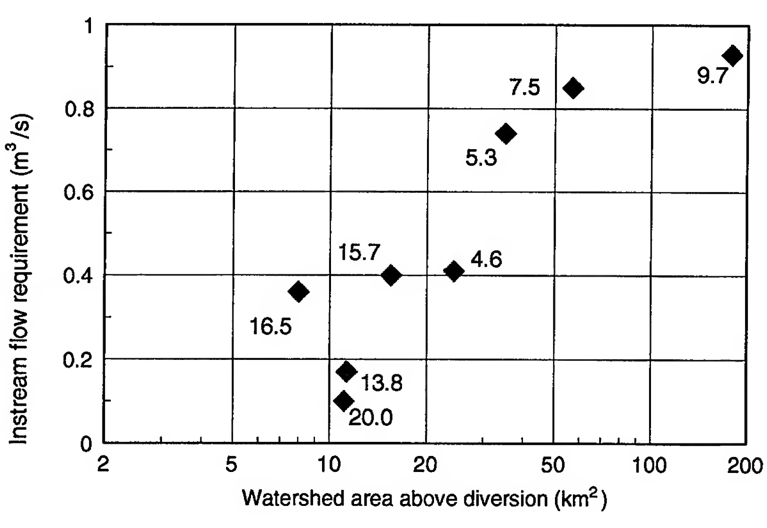

Figure 2. Instream flow recommendations derived from IFIM applications vs. watershed area and stream gradient (data points labelled with bypassed reach gradient in percent).

The correlation coefficient (r^2) for the second regression is 0.831, with F = 12.3 (Prob. > F less than 0.05).

Discussion

Our results illustrate one reason why the Tennant Method may not be a suitable standard-setting method, at least for streams in the Cascade region. **Figure 3** shows the relationship between our average IFR values and QAA, including lines for 10% and 30% of the QAA. Assuming that the IFIM applications produced a constant level of habitat protection for the Cascade streams studied, the fact that the regression line of IFR vs QAA crosses Tennant's threshold lines means that the Tennant method is not providing a constant level of protection. Since QAA is proportional to watershed size for these streams, it appears that a decreasing percentage of mean annual flow is needed to protect instream habitat as watershed size increases. For largerwatersheds, Tennant's method would call for extremely conservative IFRs compared to the IFIM, effectively withholding use of streamflows for other uses (for example, power production). The IFIM calculates habitat as a function of flow velocity and stream depth, among

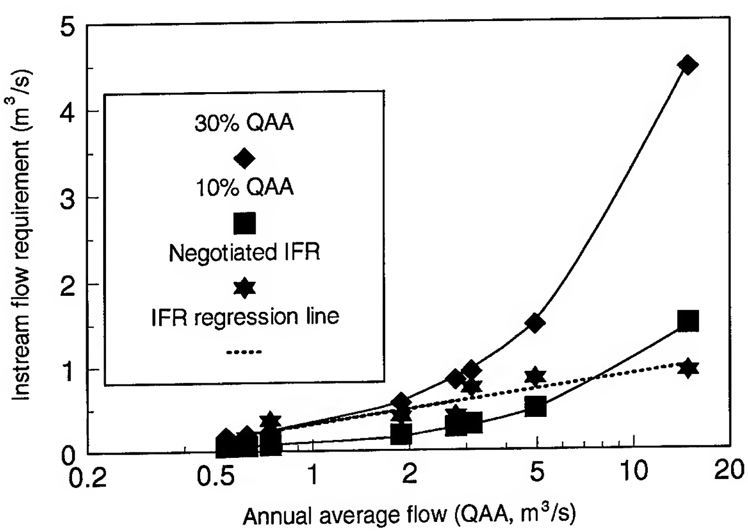

Figure 3. Instream flow requirements vs. average annual stream flow.

other factors (Bovee 1982). Velocity and depth are related to streamflow by power functions of the form a*(streamflow)^b, with the exponent (b), less than one (Thomann 1972). Therefore, the instream flow required to maintain velocity and depth, and thus habitat, does not increase linearly with annual average flow. The Tennant method assumes a linear relationship between annual average streamflow and the instream flow required to protect habitat. **Figure 3** demonstrates that, for large streams in the Skagit and Nooksack River basins, this assumption results in IFRs greater than necessary to maintain fish habitat.

The signs of the regression coefficients in equation (2) provide insight into the relationship between stream characteristics and the IFRs. Both the negotiated and predicted IFRs increase with drainage area. This is not surprising, for two reasons: average annual flow is an increasing function of drainage area; and, as discussed above IFR is an increasing function of average annual flow. The predicted IFR is also a decreasing function of gradient. This stems from the fact that higher gradient streams tend to have bedrock-controlled channels with relatively simple cross-sections. As modeled by the IFIM, velocity tends to be a monotonically increasing function of streamflow throughout a bedrock-controlled channel. Therefore, higher flows create velocities unsuitable for fish habitat in much of the channel cross-section. IFRs designed to maintain fish habitat tend to be lower for high-gradient streams than for low-gradient streams of similar size (Beecher 1992). For example, two of the study streams, Olson and Canyon Lake Creeks, have virtually the same drainage area. Both the actual and predicted IFRs are significantly lower for Olson Creek, which has a much higher gradient.

The relationships between IFIM-based IFR and watershed characteristics can be used to generate estimates of expected IFRs at unstudied sites. Estimated IFRs are calculated in three steps. If sufficient streamflow records are available, the first step is calculating QAA from them. If streamflow records are inadequate, the first step is begun by locating the proposed bypassed reach on a topographic map. From the map, both the drainage area above the proposed diversion point and the average gradient of the bypassed reach are then determined, completing the first step. The second step is calculation of the annual average IFR using the appropriate regression equation, (1) or (2) above. Finally, regardless of which regression equation is used, the low, transition, and high seasonal IFRs are ratios of 0.5, 1, and 1.5 times the regression result.

The IFR data points used in the regressions result from application of the IFIM followed by a negotiation process with reasonably consistent

objectives. Thus, the regression-based IFRs derived from these data points probably provide a consistent level of habitat protection. **Figure 3** plots the IFR data points and regression line, along with the IFRs based upon the Tennant method. Assuming that the regression-based IFRs represent a baseline of constant habitat protection, the Tennant method provides varying habitat protection depending upon annual average streamflow.

Conclusions

The regression equations presented indicate that much of the variability of IFIM IFR results can be accounted for by watershed characteristics. The indication that IFIM results are strongly influenced by watershed characteristics is encouraging, because IFRs based on our regressions are likely to provide more consistent habitat protection over a wider range of stream types than does the widely-used Tennant method.

The data are derived from streams with very similar topographic, climatic, and ecological settings. Given the strong correlation shown, other streams in the area with similar species of interest should exhibit a similar relationship between the proposed, easily measured characteristics and the results of IFIM-based instream flow negotiations.

More than one regression equation may be required to predict the results accurately. **Table 1** indicates that not all data points were based upon exactly the same habitat objective. It is possible that IFR agreements based upon maintaining existing habitat (streams denoted with NNL or MHL in **Table 1**) represent a different underlying process than those based upon "optimizing" habitat (streams denoted by POC in **Table 1**). Despite these problems, our results are quite encouraging.

Further studies of instream flow methods are needed to confirm the relationships we observed. The following list will be necessary followups to our initial analyses:

-- collection of many additional data points within the Cascades region;

-- investigation of separate regressions for IFIM results conducted under "no net loss" and "peak of curve" habitat objectives;

-- extension of the analysis to other regions; and

-- extension of the analysis to other types of fish, or other types of IFIM study subjects (for example, whitewater rafting).

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FISHN
Minimum Flow Selection Made Easy

Gary M. Franc[1]

ABSTRACT

Currently, differences of opinion exist between environmental resource agencies (Agencies) and power producers in the interpretation of Weighted Usable Area (WUA) versus flow data, as a tool for making minimum flow recommendations. WUA-flow curves are developed from Instream Flow Incremental Methodology (IFIM) studies (Ref. 1-USFWS, September 1986). Each point on a WUA-flow curve defines the usable habitat area created within a bypassed reach[2], for a specific species and life stage, due to a specified minimum flow[3] being maintained within that reach.

In an effort to standardize the use of WUA-flow data to assist in minimum flow selection, the Federal Energy Regulatory Commission (FERC) published an article entitled, "Evaluating Relicense Proposals at the Federal Energy Regulatory Commission" (Ref. 2-FERC, April 1991). This paper has subsequently become known as FARGO (named after the primary author). My paper entitled, "Fargo Incorporating Streamflow Hydrology Nuances" or FISHN will discuss an extension to FARGO.

1.0 BACKGROUND

FARGO is a general balancing approach premised on **extracting the greatest overall**

[1]Project Engineer, Niagara Mohawk Power Corporation, 300 Erie Boulevard West, Syracuse, NY, 13202. (315-428-6083)

[2] A bypassed reach is the stretch of river between a hydro development's dam and the confluence of the development's tailrace with the river. In the absence of a minimum flow, non-pool portions of a bypassed reach are typically dry during low inflow periods due to the diversion of flow through the development's powerhouse.

[3] "Minimum flow" refers to the lowest allowable flow (expressed in cfs), or inflow if inflow is less than this minimum flow, that is required to be continually discharged into a bypassed reach.

value from the water resource and is not limited to problems pertaining to just minimum flow selection. However, this paper will focus on the problem of minimum flow selection only.

In FARGO, alternatives are analyzed **incrementally** to determine the change in "power" and "non-power" values due to changes in the minimum flow. FARGO measures benefits in terms of the **WUA** created from maintaining a minimum flow. Costs are determined from annual and/or seasonal energy simulations which account for the energy lost due to diverting water into the bypassed reach. These energy losses are converted to costs by using a long-term avoided cost factor. Bar charts are used to show **incremental** changes in habitat and the corresponding **incremental** costs incurred plotted against an **incremental** increase in minimum flow. Plotting habitat and costs on a **per unit flow basis** eliminates the need to select a constant minimum flow increment and allows one to meticulously analyze specific flow ranges while cursorily investigating less critical flow ranges. Although not quoted as such in the FARGO paper, this balancing analysis has become known in the hydro industry as a "bang for the buck" approach.

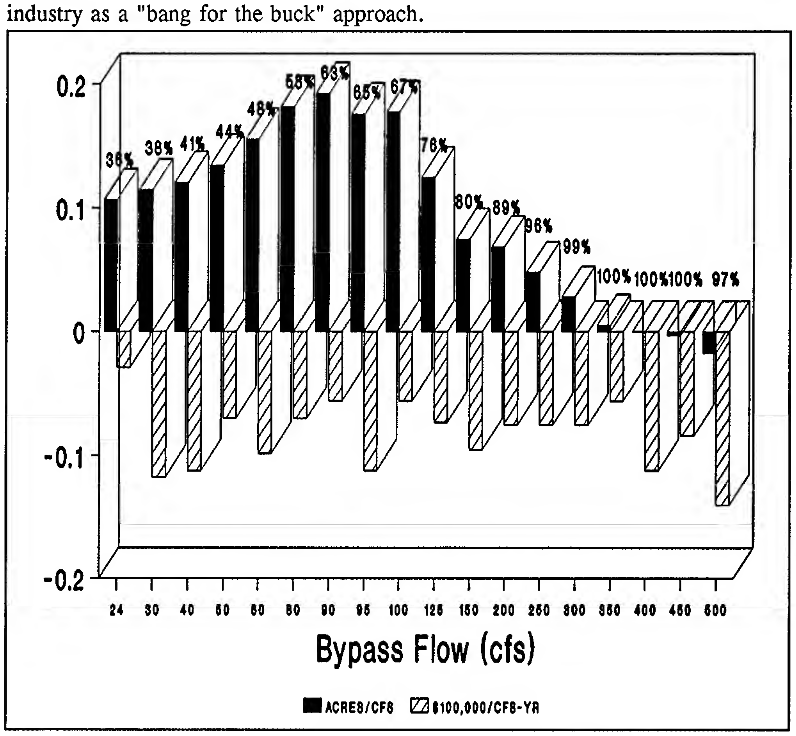

Figure 1.1 - Plot of Incremental Weighted Usable Area Versus Incremental Annual Cost Using Hypothetical IFIM Curve

For example, in Figure 1.1, the habitat increases 0.11 acres/cfs or 0.66 acres, if the minimum flow is increased from 24 to 30 cfs (6 additional cfs). Correspondingly, the annual cost increases $11,500/cfs or $69,000 over this same range, while the habitat is increased from 36 to 38 percent of its maximum attainable acreage. In effect, if one agrees to provide this additional 6 cfs, it will cost $69,000 each year to provide 0.66 additional acres. This annual cost can alternatively be expressed as $104,550/acre. **In FARGO, the decision to accept or reject this incremental increase in minimum flow must be made.** In doing so, one is intuitively assigning a value or worth to the habitat. In general, this procedure continues until an **unacceptable annual cost per acre results**. The main pitfall with this approach is in assigning the unacceptable annual cost per acre. Agencies and power producers have diametrically opposing views as to habitat value.

Another inherent limitation with FARGO, when used in minimum flow selections, is that FARGO does not adjust the WUA-flow curves resulting from the IFIM study to account for the diversity in magnitude and frequency of flows experienced within the bypassed reach over time. Accordingly, any flow analyzed in an IFIM study that exceeds the 100 percent exceedence inflow overestimates the habitat potential available, since the WUA-flow curve is not adjusted to reflect the portions of a month, season or year when inflow is less than this flow and correspondingly less habitat is available within the bypassed reach.

Also, FARGO gives no consideration to habitat resulting from monthly, seasonal or annual spillage flows that wet the bypassed reach intermittently whenever inflows exceed the hydraulic capacity of the powerplant. When Agencies request power producers to mitigate the effects on a fishery due to a reduction in the frequency of spillage at projects targeted for capacity expansion, they are actually arguing that habitat temporarily created by spillage has value. If spillage and its temporary effect on a reach's habitat are consider valid for certain species/life stages being studied, one can distinguish between the minimum flow required to mitigate impacts due to hydraulic capacity expansion and the minimum flows which represent habitat enhancement. The decision to consider intermittent habitat benefits should be made by the biological expert of the study team. How intermittent habitat is incorporated into the analysis is defined in more detail in Section 2.2.1.

Although problems exist with FARGO, the approach does add insight to habitat versus cost tradeoffs, which simplifies the balancing process required to properly select a minimum flow.

2.0 FISHN

Like FARGO, FISHN can use bar charts to analysis habitat gain versus cost tradeoffs on an incremental basis. However, FISHN performs either a "**biggest** bang" or optionally a "**biggest** bang for the buck" approach. Both approaches recommend a minimum flow **without having to assign a value to the habitat.**

2.1 SELECTION OF MINIMUM FLOW USING FISHN

FISHN uses standard hydrological and engineering practices, compares facts derived from the IFIM study results and optionally allows for habitat value adjustments based on biological considerations which are related to the percent availability of habitat. FISHN then provides graphic illustrations to help one make an informed minimum flow selection which balances resource tradeoffs. Specifically, FISHN recommends selecting a minimum flow based on one of two objectives: (1) Choose the minimum flow which maximizes the amount of WUA created per cfs ("biggest bang" option), or; (2) Choose the minimum flow which minimizes the cost per acre to provide the habitat. ("**biggest** bang for the buck" or stated conversely, "**smallest** buck for the bang" option).

Option 1 is based on a diminishing rate of return approach which completely ignores costs. It is premised on the idea that flow should continue to be allocated to the bypassed reach as long as the next increment of flow produces an equal or greater increment of habitat. Option 2 incorporates costs only as a means of determining the cost per acre of habitat. The economic viability of providing the minimum flow is not considered in the selection of the minimum flow.

2.1.1 MINIMUM FLOW SELECTION USING INCREMENTAL DATA

To illustrate how FISHN recommends a minimum flow using **incremental** data, lets revisit the data in Figure 1.1 again. The "**biggest** bang" and the "**biggest** bang for the buck" both occur when one increases the minimum flow from 80 to 90 cfs. This additional 10 cfs of minimum flow creates an additional 0.19 acres/cfs or 1.9 acres of habitat. The cost required to support this 10 cfs addition in minimum flow is $5,500/cfs or $55,000 annually. Beyond 90 cfs, each incremental increase in minimum flow results in a continually decreasing rate of habitat gained per unit of minimum flow (ie. acres/cfs). For example, increasing the minimum flow from 90 to 95 cfs creates an additional 0.175 acres/cfs or 0.875 acres of habitat. However, this rate of habitat gain per unit flow is less than the previous intervals' 0.19 acres/cfs. Also, the cost to provide this additional 0.875 acres of habitat has increased to $11,000/cfs or $55,000 annually. Simply stated, **The same amount of dollars which provides 1.9 acres of habitat in the previous interval only provides 0.875 acres in this interval.** Therefore, one should reject expanding the minimum flow beyond 90 cfs since the incremental "habitat bang" is decreasing for the same "buck". Note in this example the minimum flow is selected without having to arbitrarily assign a value to the habitat.

2.1.2 MINIMUM FLOW SELECTION USING ENHANCED DATA

To illustrate how FISHN recommends a minimum flow using **enhanced** data, lets recalculate the data in Figure 1.1 on an enhanced basis. In general, using **enhanced** rather than **incremental** values will result in the selection of the same or larger

minimum flow.

By definition, **enhanced habitat is the total proposed habitat minus the existing baseline habitat and enhanced minimum flow is the total proposed minimum flow minus the mitigated flow.** Mitigated flow is the minimum flow required to mitigate proposed adverse impacts to the existing project. Mitigated flow is discussed in detail in Section 2.2.1. For now, assume 20 cfs is required to mitigate the proposed project changes.

For each minimum flow in Figure 1.1, determine the amount of **enhanced** WUA created per unit of **enhanced** minimum flow provided to the bypassed reach. This is accomplished by dividing the enhanced WUA created by the enhanced minimum flow. Plot the above results against minimum flow as illustrated by the thick solid line in Figure 2.1. In Figure 2.1, the "biggest habitat bang" occurs for a minimum

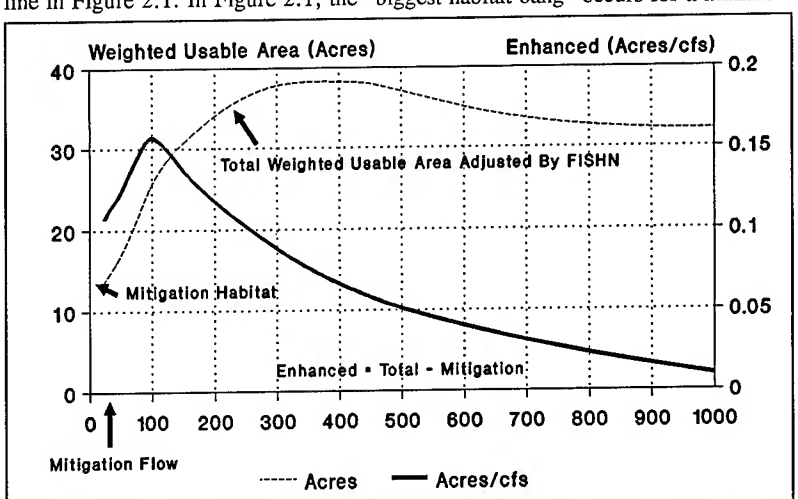

Figure 2.1 - Minimum Flow Selection Using FISHN Option 1 and <u>Enhanced</u> Data

flow of 100 cfs, since this minimum flow **maximizes the enhanced habitat on a per unit flow basis** (Enhanced (Acres/cfs)). You may recall using incremental values resulted in a recommended minimum flow of 90 cfs. At a minimum flow of 100 cfs, a total of 26 acres of effective WUA would be available (dashed line), an increase of 12.8 acres (0.16*(100-20)) above existing conditions. Also, as shown in Figure 1.1, 67 percent of the maximum possible habitat would be created.

Alternatively, the "biggest bang for the buck" minimum flow can be determined using enhanced values. For each minimum flow analyzed in Figure 1.1, determine

the associated annual cost of providing the habitat. Present worth each annual cost to the expected in-service date of the proposed project. Divide each present worth cost by the **enhanced** WUA gained for each minimum flow and plot against minimum flow. The resulting curve establishes the **minimum allowable habitat value per acre**, expressed as a one time up front purchase price for land ($/acre), which corresponds to releasing various minimum flows throughout the proposed license term (See thick solid line in Figure 2.2). This curve illustrates what the habitat value per acre must be for various minimum flows to have habitat benefits gained equal costs incurred over the life of the project. Obviously, the larger this minimum value becomes, the more risky the decision is to release the corresponding minimum flow since the chances are less likely that the benefits can justify the costs.

Assume that one recommends a minimum flow of 40 cfs. Figure 2.2 indicates the habitat value must be at least $900,000 per acre. If in actuality, the habitat is worth more per acre, a traditional benefit-cost analysis would result in a B/C ratio exceeding 1.0. If however, the habitat is actually worth less per acre, the B/C ratio would be below 1.0. If the "biggest bang for the buck" minimum flow of 100 cfs is selected (lowest value), the habitat value need only be $500,000 per acre to ensure the B/C ratio is 1.0 or greater. Accordingly, **regardless of the true worth of the habitat, selecting the minimum flow corresponding to the lowest price per acre ensures that either net benefits are maximized or net costs are minimized.**

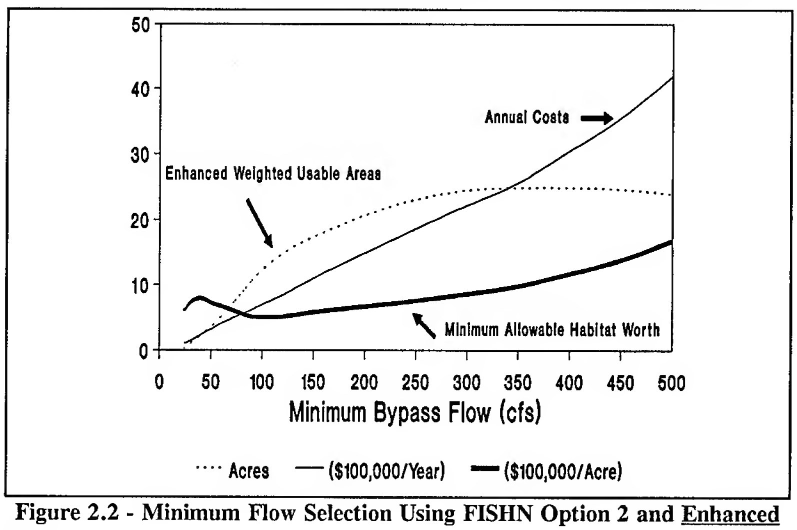

Figure 2.2 - Minimum Flow Selection Using FISHN Option 2 and Enhanced Data

Again, in this example the minimum flow is recommended without having to arbitrarily assign a value to the habitat. Only after recommending a minimum flow

of 100 cfs is the cost of releasing the minimum flow considered. Figure 2.2 indicates the cost would be $700,000 annually, over 40 years. **Only if, as a result of the economic analysis, this annual cost is found to be exorbitant, should physical modification of the bypassed reach to create the equivalent amount of habitat with a lesser minimum flow be considered.** The initial cost to modify the reach along with annual costs to maintain the reach can be compared against the benefit of regaining a portion of the minimum flow for power production. If no physical modification of the bypassed reach is economically justified, one should select the mitigated flow as the recommended minimum flow. After the minimum flow is established, a feasibility study can determine whether this required release can justify the installation of a minimum flow turbine/generator at the dam.

2.2 INCORPORATING SITE HYDROLOGY AND BIOLOGY USING FISHN

Another important aspect of FISHN is how the hydrological and biological effects at the site are incorporated into the WUA-flow curves by using the site's flow-duration curve. This allows FISHN to adjust a WUA-flow curve to account for the percent availability of inflows, to account for changes in the WUA due to intermittent spillage, and to quantify the minimum flow needed to mitigate proposed operating schemes.

FISHN requires the determination of "effective WUAs" to perform these adjustments. Effective WUA is calculated by constructing a WUA-duration curve using the WUA-flow curve from an IFIM study, the flow-duration curve for the site, an intermittent habitat value versus habitat availability curve and a selected minimum flow. This approach is analogous to performing a time-series analysis on the available habitat. **The area under the resulting WUA-duration curve represents the expected average or effective WUA that occurs for a specific minimum flow.** Effective WUA is more representative of the average habitat available for a given minimum flow within the bypassed reach, since it accounts for the diversity in magnitude and frequency of flows experienced within the bypassed reach over time.

2.2.1 MITIGATION FLOW DETERMINATION USING FISHN

FISHN determines what portion of the minimum flow is mitigation flow by comparing the effective WUA resulting from the **baseline** and **proposed** operating schemes. Differences in these effective WUAs are used to estimate **the habitat impacted, if the proposed operation results in less effective WUA than the baseline operation**, or to estimate **the habitat enhanced, if the proposed operation results in more effective WUA than the baseline operation.** In situations where impacts occur, FISHN defines mitigation flow as **the minimum flow under the proposed conditions required to create effective habitat that is equal to the effective habitat supported by the baseline conditions.**

Much debate as to the definition of **"baseline conditions"** has surfaced in recent years. Some Agencies argue pre-existing project or natural conditions should define the "baseline conditions" whereas, power producers argue the current operation of the facilities should be considered "baseline conditions". Recently, FERC has taken a stand more in agreement with the power producer's viewpoint. Some Agencies even adhere to the power producer's viewpoint. In the "Hydropower Policy of the USFWS" (Ref. 3-USFWS, July 1988), the goal of **mitigation is defined as maintaining existing habitat value.** Mitigation is distinguished from enhancement, which is defined as **"... measures that will improve fish and wildlife resources beyond the level that exists at the time of the application ...".**

Once baseline and proposed conditions are defined, biological expertise is recommended for assigning partial value to the habitat created from intermittent spillage flow. This value is expressed as a fraction of the full value that is assigned to habitat available continually. Figure 2.3 is a graph depicting three different ways of assigning partial value or worth to habitat available intermittently, along with a plot of the hypothetical "true" relationship.

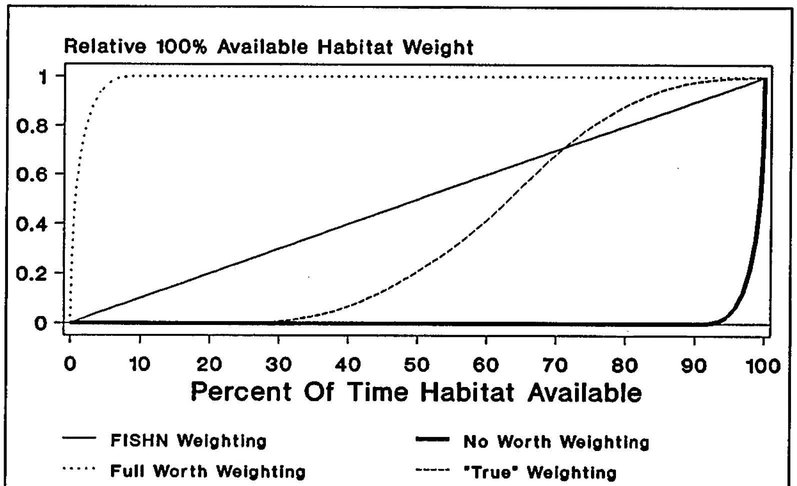

Figure 2.3 - Relative Weighting Options For Intermittently Available Habitat

WUA-flow data resulting from IFIM studies represents one extreme and assigns no value to intermittently available habitat. This is shown by the **thick solid** line. A time-series analysis represents the other extreme and assigns full worth to intermittently available habitat. This is shown by the **dotted** line. The default option in FISHN is to assume habitat value is directly proportional to the amount of time the habitat is available, as shown by the **thin solid** line. This assumption represents

an equal consideration of the two extremes depicted in Figure 2.3. The amount of time the habitat is available is determined from the flow-duration curve for the site and corresponds to the percent exceedence value of the river flow.

A more accurate relationship is likely to be a function of spillage frequency, species, life stage and time of year, as depicted by the **dashed** line. Biological expertise should be solicited in defining this relationship.

To illustrate how intermittent habitat is incorporated into a WUA-duration curve to determine effective habitat, assume a hypothetical project with one 3500 cfs turbine is required to discharge 100 cfs within the bypassed reach at all times (baseline conditions). Suppose the project is experiencing an inflow of 6000 cfs, which corresponds to the 20 percent exceedence flow. The project is therefore passing 3500 cfs through the turbine and tailrace, releasing a minimum flow of 100 cfs within the bypassed reach and passing an additional 2400 cfs over the spillway. This scenario results in a total of 2500 cfs within the bypassed reach.

Suppose 2500 cfs temporarily creates 11 acres of WUA within the bypassed reach and that when only 100 cfs is passing through the bypassed reach, 1.0 acres of WUA is supported. The adjusted WUA on the WUA-duration curve for a 20 percent exceedence would equal the continuously supported habitat plus the partial worth of the temporarily supported habitat. In equation form this equals $1.0+(11.0-1.0)(0.20)$ or 3.0 acres. This adjusted WUA results in one point on the WUA-duration curve (See Baseline Habitat curve in Figure 2.4).

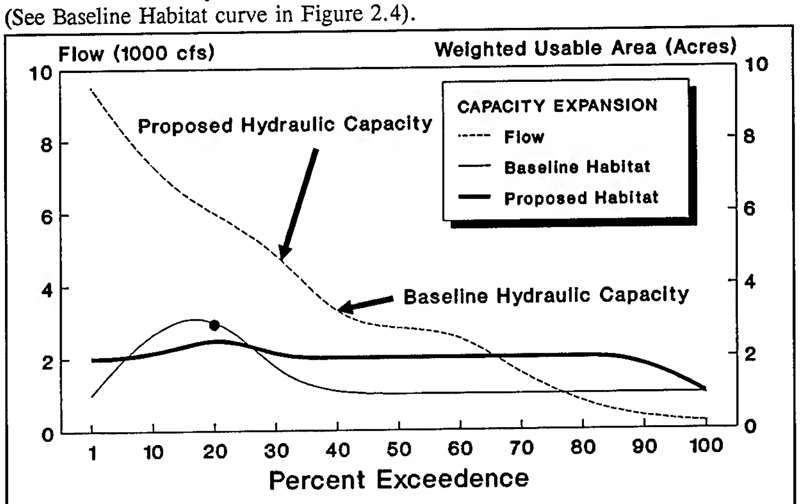

Figure 2.4 - Construction of Weighted Usable Area-Duration Curves To Determine Effective Weighted Usable Area For Given Minimum Flow

FISHN uses this process over-and-over again for percent exceedence values ranging from 0 to 100 percent to develop the baseline habitat duration curve **(thin solid line)**. The curve shows that for baseline conditions, the WUA within the reach will be 1 acre about 60 percent of the time. Above the 40 percent exceedence inflow, spillage into the bypassed reach occurs. FISHN assigns a partial value to the habitat temporarily supported by the spillage. This value becomes increasingly less valuable as the spillage becomes less frequent, until the value of the intermittent habitat is negligible at the 1 percent exceedence flow. **The area under this WUA-duration curve represents the effective WUA assuming baseline conditions.**

The **thick solid line** represents the WUA-duration curve for proposed conditions. Proposed conditions are to increase the hydraulic capacity from 3500 to 4500 cfs and the minimum flow from 100 to 500 cfs. A 500 cfs minimum flow will support 2 acres of habitat. The curve shows that for this scenario, the WUA within the reach will be 2 acres about 55 percent of the time (85 to 30 percent exceedence range). Above the 30 percent exceedence inflow, spillage into the bypassed reach occurs. FISHN assigns a partial worth to the habitat temporarily supported by the spillage. This intermittent habitat is comparably less valuable then for the baseline condition and becomes increasingly less valuable as the spillage becomes less frequent. Below the 85 percent exceedence flow, less than 500 cfs is available with the minimum flow and corresponding habitat somewhat reduced. **The area under this WUA-duration curve represents the effective WUA assuming proposed conditions.**

If the effective habitat area for proposed conditions is less than for baseline conditions, additional minimum flow must be allocated to the reach to mitigate impacts. A trial-and-error process is used by FISHN until a minimum flow is found, such that the two effective areas are the same.

2.2.2 ADJUSTMENT OF WUA-FLOW CURVES USING FISHN

This same procedure of constructing WUA-duration curves is used to develop the effective WUA-flow curves used by FISHN. The process is repeated up to twenty times for a range of assumed minimum flows. A hypothetical example of this adjustment to a WUA-flow curve (Figure 2.6) resulting from incorporation of a site's hydrology (Figure 2.5) and biology (Figure 2.3) is shown below.

The adjusted WUA-flow curve in Figure 2.6 resulted from constructing WUA-duration curves and determining the corresponding area under each curve for assumed minimum flows of 0, 100, 200, 300, 400, 500, 600, 700, 800, 900 and 1000 cfs. This curve shows the habitat is lower than the habitat from the unadjusted curve once the minimum flow exceeds 500 cfs. This is due to the inability of the project to maintain a 500 cfs minimum flow continually. From Figure 2.5, inflows below 500 cfs will occur approximately 12 percent of the time, resulting in lower bypass releases and correspondingly less WUA. This **"habitat discounting"**

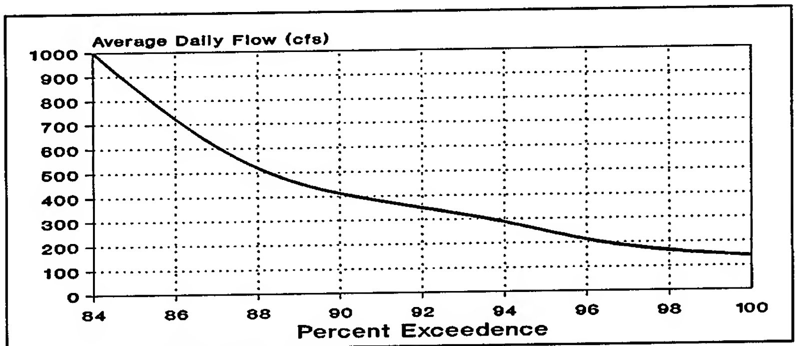

Figure 2.5 - Hypothetical Flow-Duration Curve

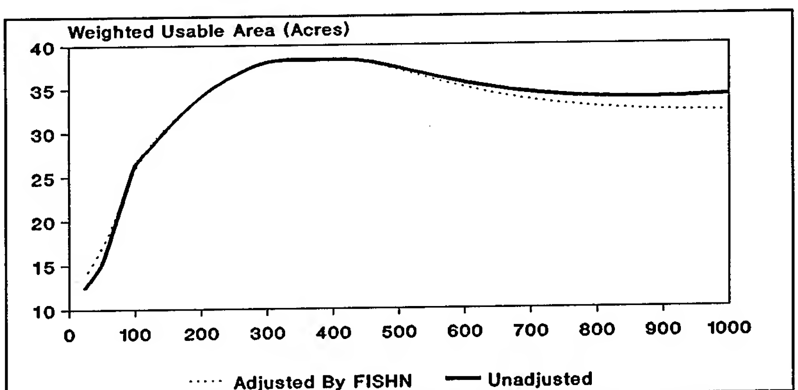

Figure 2.6 - FISHN Adjustment to IFIM Supplied Weighted Usable Area Versus Flow Curve Based on Hydrology (Figure 2.5) and Biology (Figure 2.3)

increases as the minimum flow requirement gets larger because the inability to maintain the larger flow releases is also increasing.

Also, the FISHN adjusted curve shows higher habitat areas than the unadjusted curve below 100 cfs. This adjustment is attributed to the partial habitat value associated with intermittent spillage. If intermittent habitat value were not considered, the two curves in Figure 2.6 would be identical within this range of minimum flows (0 to 100 cfs).

2.3 FISHN SPREADSHEET PROGRAM FOR PC's

A stand-alone FISHN spreadsheet program is available for PC's to perform the numerous calculations required and to plot results. The FISHN approach can be

performed on either an annual, seasonal or monthly basis. For each season analyzed, FISHN must be supplied with flow-duration curve(s), maximum and minimum hydraulic capacities and WUA-flow curve(s) for both the **baseline and proposed** operating schemes. Also, for each minimum flow to be analyzed, estimates of the corresponding energy losses can be supplied or optionally determined by the software.

3.0 CONCLUSIONS

FARGO, originally conceived and promoted by FERC, is an approach which improves and simplifies the balancing process required to properly select a minimum flow. FISHN is a logical extension of FARGO which allows for minimum flow selection without having to assign a value to the habitat. FISHN uses standard hydrological and engineering practices, allows for biological considerations, compares facts derived from the IFIM study results and provides graphic illustrations to help one make an informed minimum flow selection which balances resource tradeoffs.

FISHN incorporates the hydrology at the site into the analysis by using the site's flow-duration curve, thereby resulting in adjustments to the WUA-flow curves generated from IFIM studies. These adjustments account for the percent availability of inflows and for changes in the WUA due to intermittent spillage. This allows FISHN to quantify the minimum flow needed to mitigate proposed operating schemes.

4.0 CONVERSIONS

TO CONVERT FROM	TO	MULTIPLY BY
Acres	Hectares	0.4047
Cubic feet/second (cfs)	Cubic meters/second (cms)	0.02832
Acres/cfs	Hectares/cms	14.29025

5.0 BIBLIOGRAPHY

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Comparison of Hydroacoustic and Net Catch Estimates of Fish Entrainment at Tower and Kleber Dams, Black River, Michigan

by

Samuel V. Johnston[1]
Bruce H. Ransom[1]
Joseph R. Bohr[2]

Abstract

As part of Wolverine Power Supply Cooperative's FERC licensing process, dual-beam hydroacoustics (sonar) and tailrace net sampling were used to evaluate fish entrainment at Tower and Kleber dams on the Black River in Michigan.

At each dam, one hydroacoustic transducer was installed at the center of one turbine unit, downstream of the trash racks. Transducers were located near the water surface and aimed downward. Fish were entrained at this point, and virtually no fish were observed holding against the current.

Hydroacoustic data collection occurred biweekly at each dam, 3 d/week, 24 h/d, for a 12-mo period in 1990 and 1991. Hydroacoustic data analysis was conducted in real time, using echo counting and target tracking techniques. Fish tracking and counting was accomplished using an HTI *Model 300 Digital Echo Processor*.

A tailrace net covering a draft tube exit captured fish passing through the turbine unit being acoustically monitored. Net sampling was conducted once per month for approximately 24 h. The results presented below based on hydroacoustic data collected concurrently with net catch data.

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1 Senior Fisheries Biologists, Hydroacoustic Technology, Inc., 715 NE Northlake Way, Seattle, WA 98105-6429; (206) 633-3383

2 Fisheries Biologist, Private Consultant, Joseph Bohr, 7521 South Forest Hill Road, St. Johns, MI 48879; (517) 626-6981

Hydroacoustic estimates of entrainment reliably predicted concurrent net catches. There was no statistically significant difference between the hydroacoustic and net catch estimates of fish entrainment at either dam. For both dams, regressions between hydroacoustic and net catch estimates were highly significant, and there was no significant difference between the regression line and a 1:1 line.

Potential sources of variability are discussed. For hydroacoustic estimates, this variability included sample variability resulting from the ensonified beam not covering 100% of the intake opening. Sources of variability in the net catch estimates included occasional infiltration of tailrace fish into the net, and incomplete flushing of fish from the draft tube of the turbine before net sampling.

Introduction

As part of the FERC licensing process, Wolverine Power Supply Cooperative contracted fixed-location dual-beam hydroacoustic (sonar) and tailrace net sampling to evaluate fish entrainment at Tower and Kleber dams on the Black River in Michigan. Entrainment at each dam was estimated biweekly for 12 months in 1990 and 1991.

In addition to comparing monthly net catch results with concurrent hydroacoustic estimates of fish entrainment, the primary objectives of the hydroacoustic evaluations were to estimate fish entrainment, and the temporal (diel), vertical, horizontal, and target strength (i.e., acoustic size) distributions of entrained fish. Detailed results are available in Johnston and Ransom (1991).

Site Description

Tower and Kleber dams are run-of-the-river dams located on the Black River near the town of Tower, Michigan. The powerhouse at Tower Dam is 35 ft long and 32 ft wide, and contains two 30 inch Type "Z" vertical-shaft turbine generator units. Each unit has a rated generating capacity of 280 kW for a total project capacity of 560 kW. The total hydraulic capacity of the powerhouse is 360 cfs, and average annual inflow is approximately 270 cfs. The spacings between the trashrack bars immediately upstream of the headgate opening are 1 inch (2.5 cm). The Tower Pond has a storage capacity of 620 acre-ft at normal elevation of 722.1 ft, with an average head of 20 ft.

Kleber Dam lies 2.9 mi downstream from Tower Dam. The powerhouse is 42 ft long and 40 ft wide and contains two vertical-shaft Kaplan turbine generator units. Each unit has a generating capacity of 600 kW for a total project generating capacity of 1200 kW. The total hydraulic capacity of the powerhouse is 400 cfs. The trashrack immediately upstream of the penstock intake at Kleber Dam has openings of 3 inches (76 mm) between the vertical bars. Kleber Pond has a storage capacity of 3,000 acre-ft at the normal elevation of 701.1 ft, with an average head of 44 ft.

In decreasing order, the primary fish species captured at Tower Dam were blackside darter, rock bass, brown bullhead, bluegill, white sucker, brook trout, common shiner, and northern pike. Primary fish captured at Kleber Dam were yellow perch, bluegill, rock bass, white sucker, pumpkinseed, brown bullhead, and brook trout.

Methods

Hydroacoustic techniques are described in detail by Albers (1965) and Urick (1975). Fixed-location hydroacoustic techniques use one or more transducers placed on a fixed structure (e.g., the wall of a turbine intake, or a trashrack), sampling fish as they pass through the ensonified acoustic beam(s) (Ransom 1991). These fish produce characteristic traces on chart recorder echograms, and the returning acoustic signals can be processed to produce estimates of fish entrainment rates into the monitored turbine intakes.

The hydroacoustic system used at Tower and Kleber dams included the following:

- *Simrad/HTI EY200 Scientific Echo Sounder*
- HTI *Dual-Beam Transducers*
- HTI *Dual-Beam Transducer Cables*
- HTI *Model 300 Digital Echo Processor* (with Real-Time *TRACKER* software)
- HTI *Model 404 Digital Chart Recorder*
- Micro-Computer (386)
- Digital Audio Tape (DAT) Data Recording System
- Oscilloscope

The *EY200 Echo Sounder* operated at a frequency of 200 kHz. Each dual-beam transducer produced both 6° and 15° beams.

The echo signals were relayed to the HTI *Model 300 Digital Echo Processor (DEP)*, a computer-based dual-beam processor that processes the acoustic signals and detects individual fish. *TRACKER* software was used to track individual fish targets. These detections were then extrapolated to obtain total fish entrainment rates. The *DEP* also estimated fish acoustic size (i.e., target strength) for each fish monitored.

The echo signals were permanently displayed on the paper recordings (echograms) produced by the *Model 404 Digital Chart Recorder*. The echogram provided a permanent record of all targets which had an amplitude greater than a predetermined threshold. In addition, the echogram also contained information regarding the fish trace range and trace type. The trace type information was used to interpret fish behavior and direction.

The data taping system both provided records of the conditions and events as they appeared in the field. An oscilloscope was used for monitoring the system operation.

One transducer was installed at each of the two dams. The transducers were mounted in a manner that maximized the sampling coverage, minimized acoustic interference, and minimized the effect of fish behavior on the interpretation of the data. Each transducer was mounted at the center of the turbine intake it monitored, downstream from the trashracks, at the opening to the intake. The transducer mounting locations and orientations are shown in Figures 1 and 2 for Tower Dam, and Figures 3 and 4 for Kleber Dam. Water velocities in the area monitored were fast enough that fish small enough to pass through the trashrack would be entrained in the flow and pass through the turbine unit. Virtually no fish were observed holding against the current.

The complete dual-beam hydroacoustic system used was calibrated in the laboratory prior to and after use in the field. On-site, in-field calibrations were conducted to periodically insure that parameters had not changed. According to the pre-data collection calibration of the single-beam echo sounder system used, as well as estimates of the minimum size fish of interest (i.e., 50 mm (2 inches) in length), the mark threshold was set at -51 dB during data analysis and on the chart recorder echogram.

Hydroacoustic data were collected biweekly at each dam, 3 d per week, 24 h/d, from April 1990 to April 1991.

A tailrace net was deployed at Tower and Kleber dams, in the draft tube outlet of the turbine intake being sampled hydroacoustically. The net sampled the entire outlet. The net was constructed of 1/4 inch (6.3 mm) bar knotless nylon webbing. It was 10 ft by 15.5 ft (3.0 m by 4.7 m) at the mouth, 20 ft (6.1 m) long, and tapered to a detachable framed live box. After an initial "flush" sample, the net was emptied as often as was deemed necessary to minimize fish stress and mortality.

Net catches were compared with the results from concurrent hydroacoustic sampling. The tailrace net was deployed for approximately 24 h at each dam, once per month. A Wilcoxon Sign Test was used to test for statistically significant differences between the paired data sets. Least square regression techniques were used to examine the relationship between net catches and hydroacoustic estimates of fish entrainment.

Data were analyzed using echo counting and target tracking techniques. Fish less than 50 mm (2 inches) were excluded. Due to the relatively low density of targets (i.e., non-overlapping), echo integration techniques were not required, or desired. Since studies of this nature involve large volumes of data, microcomputers were used for data storage and analysis. Data and results were transferred to HTI's Seattle offices weekly.

Each transducer was sampled continuously. Once target echo data were acquired, the fish were tracked by the *DEP*. Hourly fish entrainment estimates were then calculated as follows.

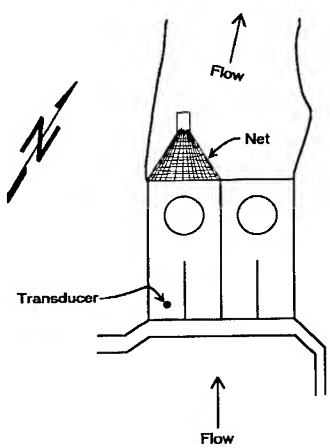

Figure 1. Plan view of Tower Dam powerhouse showing location of hydroacoustic transducer and tailrace net.

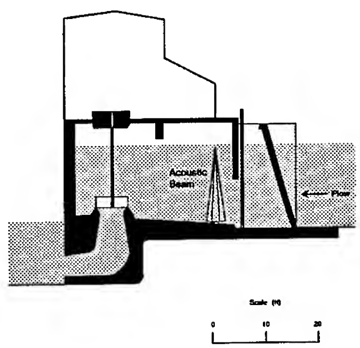

Figure 2. Cross-section of Tower Dam powerhouse showing location of transducer and sample volume.

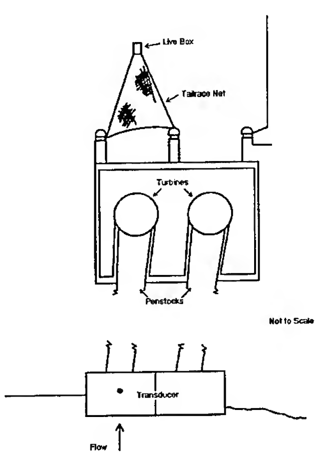

Figure 3. Plan view of Kleber Dam showing location of hydroacoustic transducer and tailrace net.

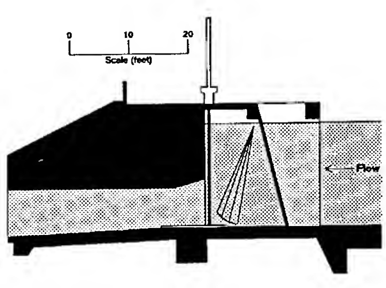

Figure 4. Cross-section of Kleber Dam intake showing location of transducer and sample volume.

Since the ensonified acoustic beam did not cover the entire cross-section of the area of the intake, not all of the fish passing into the intake were detected. The total number of fish passing each hour was estimated by weighting each fish detection by the proportion of the area sampled (at the range of the detection). Each fish detection was weighted by the ratio of the intake width to the effective diameter of the acoustic beam at the range of a detected target. Since the acoustic beam is conical, each individual fish detection was weighted as follows:

$$W_f = \frac{I_w}{2\,R \tan(\alpha/2)}$$

where

W_f = the estimate of fish entrained.
I_w = turbine intake width.
R = range of the fish from the transducer.
α = the transducer beam width (a function of beam pattern, individual fish target strength, and equipment parameters).

All the weighted fish were then summed to estimate the total number of fish entrained at the turbine monitored.

The dual-beam technique was used to estimate acoustic target strength. A detailed explanation of this technique is given by Ehrenberg (1978, 1984). Love (1977, 1981) has developed widely used relationships for relating acoustic target strength to the length of fish. Target strength results (in decibels) were converted to predicted fish lengths following these methods.

Tower Dam Results

At Tower Dam, 65% of the fish monitored hydroacoustically were 89 mm or less in length, and 84% were 100 mm or less. Only 3% of all fish monitored were over 200 mm.

The largest fish monitored hydroacoustically were 26 cm. The spacings between the trashrack bars immediately upstream of the transducer were 1 inch (2.5 cm), and precluded large fish from entering the turbine intakes. Most of the fish sampled in the tailrace net were also small (82% < 200 mm). Of the total net catch, 6% were 30 cm or greater. Given the close spacings of the trashrack bars (1 inch (25 mm)), it is likely that the largest fish were either residents in the turbine's draft tubes, or were able to infiltrate into the tailrace net from below the dam.

Table 1 presents the results of hydroacoustic monitoring and concurrent tailrace netting at Tower Dam. There was no statistically significant difference between hydroacoustic and net catch estimates of fish entrainment ($p > 0.05$, $n = 12$). A highly significant relationship was found between net catches and hydroacoustic estimates of entrainment ($Y = -11.243 + 0.961X$; $R^2 = 0.78$, $p = 0.0001$, $n = 12$) (Figure 5). There is no significant difference between the regression line and a 1:1 line.

Table 1. Comparison of hydroacoustic and net catch results for Tower Dam.

Date	Total Net Hours	Hydroacoustic Estimate	Net Catch
Apr 28-29	23.0	218	261
May 18-19	23.7	87	32
Jun 14-15	24.6	261	181
Jul 19-20	24.5	14	61
Aug 15-16	24.8	29	22
Sep 7-8	24.0	77	16
Oct 5-6	25.6	77	45
Nov 8-9	26.0	86	28
Dec 6-7	23.2	33	4
Jan 31-Feb 1	24.2	7	11
Feb 28-Mar 1	24.1	0	4
Apr 5-6	26.2	202	249
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Mean (n = 12)		91	76

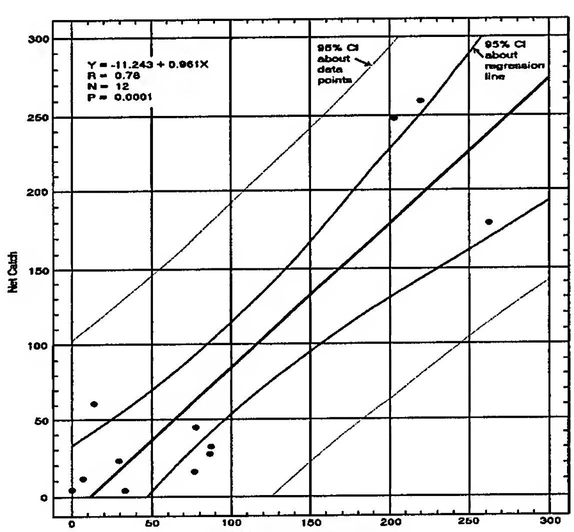

Figure 5. Linear regression of hydroacoustic entrainment estimates versus net catch for Tower Dam.

Kleber Dam Results

Like at Tower Dam, most of the fish entrained at Kleber Dam were small. Over 78% of the fish were 80 mm or less in length, and 93% were 100 mm or less. Only 1% of all fish monitored hydroacoustically were 160 mm or greater. Most of the fish captured by the tailrace net were also small, with 96% at 200 mm or less length. Less than 1% of the captured fish were 300 mm or greater. Given the spacing of the trashrack bars (3 inches (76 mm)), large fish would not be expected to pass voluntarily through the trashrack and into the intake area.

Table 2 presents the results of concurrent hydroacoustic monitoring and tailrace netting at Kleber Dam. There was no statistically significant difference between hydroacoustic and net catch estimates of fish entrainment ($p > 0.05$, $n = 11$). Using all 12 data points, a statistically significant relationship was found between net catches and hydroacoustic estimates of entrainment ($Y = \log 2.261\ X^{0.614}$, $R^2 = 0.42$, $p = 0.003$, $n = 12$). There is no significant difference between the regression line and a 1:1 line.

There is evidence that the first sample, the one with the largest difference between hydroacoustic and net samples, suffered from considerable infiltration into the sample net from fish in the tailrace. A more representative correlation between net and acoustic estimates excludes this data point. Using the remaining 11 data points, a highly significant relationship was found between net catches and hydroacoustic estimates of entrainment ($Y = \log 2.040\ X^{0.608}$; $R^2 = 0.64$, $p = 0.003$, $n = 11$) (Figure 6). This high level of significance indicates a very close correlation between the two estimates, indicating that the hydroacoustic estimates of entrainment were good predictors of concurrent net catch.

Discussion

Hydroacoustic target strength measurements at both projects indicated that most entrained fish were small. Fish 100 mm or less in length made up 84% of all fish entrained at Tower Dam, and 93% of all fish at Kleber Dam. Length frequency distributions from fish captured in net catch samples were similar to hydroacoustic length distributions.

Hydroacoustic estimates of entrainment reliably predicted concurrent net catches. There was no statistically significant difference between the hydroacoustic and net catch estimates of fish entrainment at either dam. Highly significant relationships were found between net catches and hydroacoustic estimates of entrainment at both dams.

Differences between estimates can be attributed to several factors. Some sampling variability was expected in the hydroacoustic estimates, since the ensonified beam did not cover 100% of the intake opening. This variability averaged out over time (i.e., for some samples the entrainment estimate was a little low, and for some samples it was a little high). Indeed, this is indicated by the highly significant statistical relationship between net catch and acoustic entrainment estimates.

Table 2. Comparison of hydroacoustic and net catch results for Kleber Dam.

Date	Total Net Hours	Hydroacoustic Estimate	Net Catch
Apr 27-28	23.0	110	2662
May 17-18	23.7	140	150
Jun 15-16	24.6	796	126
Jul 20-21	24.5	238	166
Aug 16-17	24.8	314	280
Sep 6-7	24.0	579	579
Oct 4-5	25.6	283	764
Nov 9-10	26.0	123	277
Dec 7-8	23.2	43	77
Feb 1-2	24.2	8	48
Mar 1-2	24.1	38	25
Apr 4-5	26.2	7	17
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Mean (n = 11)		234	228

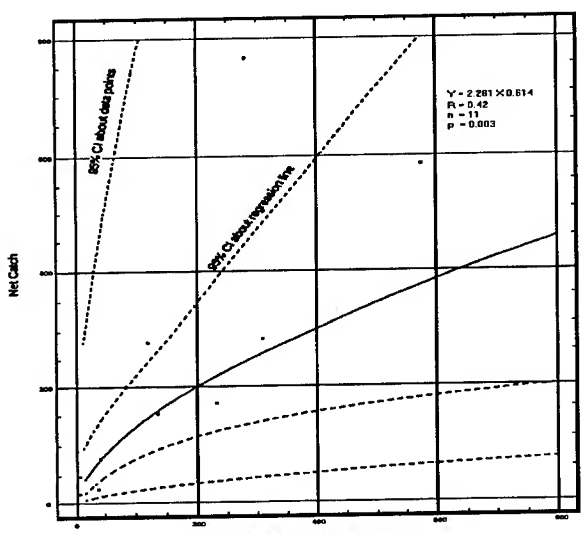

Figure 6. Linear regression of hydroacoustic entrainment estimates versus net catch for Kleber Dam.

While tailrace netting was conducted to the highest practical standards, there is also variability associated with the net catch estimates. Variability resulted from occasional infiltration of tailrace fish into the net, as was the case with the first net sample at Kleber Dam. Also, larger fish may not have been completely flushed from the draft tube of the turbine before actual test sampling began.

These factors considered, the estimates of net catch and acoustic entrainment compared very well, as indicated by the near 1:1 slope of the regression lines.

Acknowledgements

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HYDRAULICS OF A NEW MODULAR FISH DIVERSION SCREEN

T.C. Cook[1], E.P. Taft[2], G.E. Hecker[3], C.W. Sullivan[4]

ABSTRACT

To prevent fish from passing through turbines, EPRI has developed a new Modular Inclined Screen (MIS) concept. Various laboratory flow facilities have been used to test the MIS for fish screening efficiency and flow characteristics. Data from these tests show high fish screening efficiencies and favorable hydraulics for a wide range of approach velocities.

INTRODUCTION

An environmental issue for some hydroelectric plants is the facility's effect on fish which pass through the hydraulic turbines. Since 1985, the Electric Power Research Institute (EPRI) has been conducting research to develop and evaluate structural and behavioral technologies to prevent fish from passing through hydroelectric turbines. One component of these efforts has been to develop a new fish screening concept is called the Modular Inclined Screen (MIS). The modular design provides flexibility for application at many types of intakes, particularly for those plants without penstocks, and for a range of flows by varying the number of modules. It is being designed from hydraulic and fisheries viewpoints, which has resulted in a device which limits hydraulic losses while not harming fish. This paper will discuss the hydraulic testing conducted, while another paper at this conference discusses the biological test results (Winchell et al. 1993).

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1) Senior Hydraulic Engineer, Stone & Webster Environmental Services, 245 Summer Street, Boston, MA 02210 (617) 589-8728
2) Program Manager, Stone & Webster Environmental Services
3) President, Alden Research Laboratory, Inc.
4) Program Manager, Electric Power Research Institute

DESCRIPTION OF MIS

One MIS consists of a streamlined entrance with a trash rack, dewatering stop log slots, a wedge wire screen set at a shallow angle to the flow, and a bypass for diverting fish to a transport pipe or holding facility. The screen is set on a pivot shaft so that it may be tilted and thus cleaned via backflushing. The module is

completely enclosed and is designed to operate at water velocities from 2 to 10 ft/sec (0.6 to 3.1 m/sec). Conceptual plan and section views of the MIS are shown in Figure 1 of Winchell et al. 1993.

The intake of the module is rectangular in shape, and incorporates curves to minimize flow separation. The distance from the entrance to the screen is kept short to reduce construction cost of the module. The screen is a plane of commercially available 50% porosity wedge wire, set at an angle of 10° to 20°. The screen support structure under the leading (upstream) edge of the screen is depressed below the floor to provide a smooth transition from the floor to the screen. The upper (downstream) end of the screen is held vertically in place, against the bottom of the bypass structure, using cables. The upper end of the screen can be lowered to the floor, allowing water to flow backwards through the screen for cleaning.

The screen support system consists of 5 equally spaced beams running longitudinally under the full length of the screen, and the beams are fastened by cylindrical cross supports and angled tensioners. The screen is welded to the support system.

TEST FACILITIES

To develop details of the MIS concept, and to test its hydraulic and fish screening performance, three flow facilities were used. First, a small flume was used to study local velocity variations near the inclined wedge wire screen and various possible support bars manufactured integrally with the screen. This flume, with one clear side, allowed the use of a Laser Doppler Anemometer (LDA) system to quantify velocities. Losses for the screen were also measured.

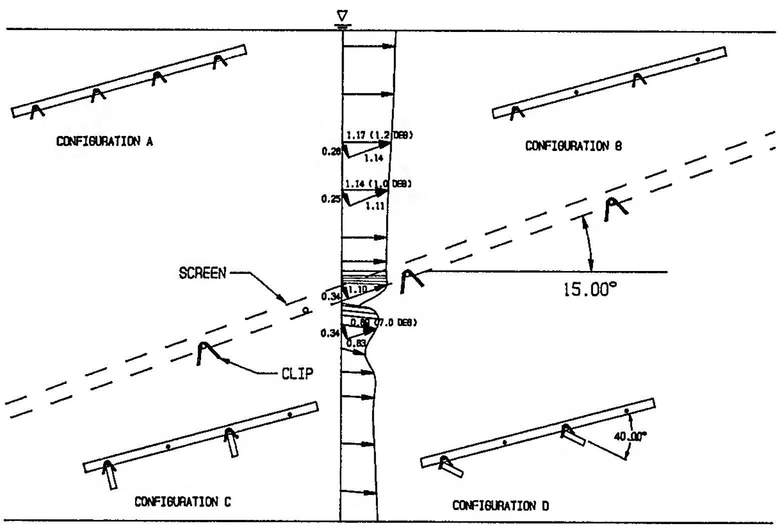
A second test facility was constructed as a 1:6.66 scale hydraulic model of the entire MIS to study overall flow characteristics. Studies included entrance effects, overall (through flow) head losses, bypass configurations and head losses, and longitudinal and lateral velocity profiles above the screen. This test facility had a closed loop flow system. It consisted of a concrete sump from which water was pumped to a plexiglass flume, through the MIS model installed in the flume, and returned back to the sump. Tests were conducted at reduced (Froude scaled) flows and velocities.

The third test facility constructed was a 1:3.33 version of the entire MIS, primarily to evaluate fish reaction, diversion efficiency, and any fish damage and delayed mortality. Hydraulic testing was performed at actual (prototype) velocities and the data were used to check test results from the smaller hydraulic model. The facility was constructed in a 7 ft (2.1 m) deep by 6 ft (1.8 m) wide steel flume in which a plexiglass viewing area was installed. The flow system was also a closed loop, driven by a marine bow thruster mounted inside a 4 foot (1.2 m) diameter return pipe, and the flow capacity exceeded 100 ft^3/sec (2.83 m^3/sec). The maximum velocity approaching the screen inside the MIS was about 10 ft/sec (3.05 m/sec).

SIMILITUDE

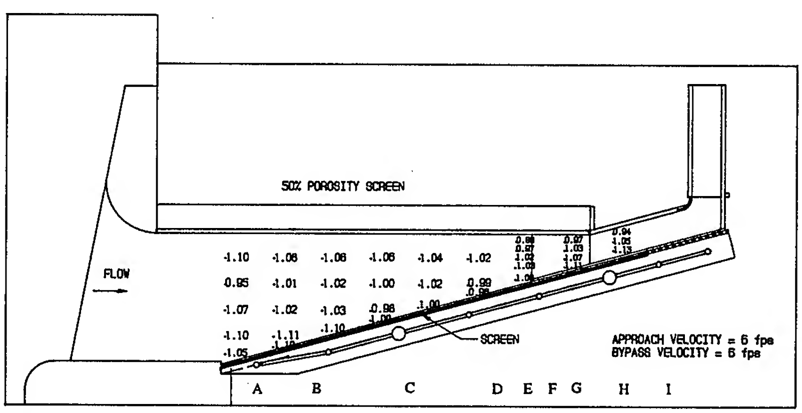
The small flume experiment was considered to be a 2D section of the actual screen approximately half-way along the MIS screen, so that the upstream and downstream boundary conditions, which could change the approach and through flow directions, were not of concern. Similarly, velocity measurements were made near the middle of the test screen to avoid flume bottom and free surface boundary effects. An actual screen panel 6 inches (0.15 m) wide was used at various angles with four support configurations as shown in Figure 1 in the 12 inch (0.3 m) deep flume, and velocities and head losses are reported as tested, without any scaling.

In contrast, the 1:6.6 model was a geometrically scaled hydraulic model of the MIS design as presently envisioned, and was operated at reduced velocities according to the usual Froude scale similitude. Reported velocities have been scaled up to the prototype by $\sqrt{6.6}$. The only item of note is the similitude of the wedge wire screen. Considering the large number of wedge wires and their negligible relative size in the prototype (and model), scaling the wire size was not considered important compared to having a model screen which has the same head loss coefficient as the prototype screen, thus producing the same effects as the flow pattern. This may be achieved with a model screen of the same geometry and porosity (assuming negligible viscous effects or sufficiently high model Reynolds numbers), and the easiest means to this end was use of the actual screen. However, all members of the support frame holding the screen and allowing rotation were modeled to scale, the unstreamlined shape and model Reynolds numbers being sufficient to simulate loss coefficients. In contrast, the lateral support bars manufactured integrally with the screen were considerably smaller than the support frame members, but considerably larger than the wedge wires, so the choice of how to scale the bars was not obvious. Since Configuration D (Figure 1) was preferred, and since: a) the loss coefficient of Configurations D and B were similar, and b) scaling the bars would result in their negligible size within the "U" clips of the screen; therefore, Configuration B was used in the hydraulic model to simulate Configuration D.



VELOCITY COMPONENTS NORMALIZED WITH APPROACH FLOW VELOCITY
USING 50% POROSITY SCREEN IN THE SMALL FLUME

FIGURE 1



HORIZONTAL COMPONENTS OF VELOCITIES
NORMALIZED WITH APPROACH FLOW VELOCITY
1:6.66 HYDRAULIC MODEL

FIGURE 2

The 1:3.3 facility was treated as a reproduction of the MIS, although of smaller size, as its primary purpose was testing with fish. Although all dimensions other than the screen (Configuration B) were reduced by a factor of 3.3, actual velocities were used in all tests in this facility. Velocities and head losses are, therefore, reported as measured without any scaling.

INSTRUMENTATION

Velocity measurements in the small flume were made by commercial LDA equipment. Velocity measurements in the hydraulic model (1:6.6 scale) and in the fish test facility (1:3.3 scale) were made with a United Sensor 2-D (three-hole) pitot meter. The probe was calibrated in a facility with a known velocity and angle using two differential pressure (DP) cells and a computer based data acquisition system. The same system was used for the actual measurements. Data reduction programming produced the speed and direction of the velocity relative to horizontal. From this information, velocity components perpendicular and parallel to the screen were calculated using basic geometric relationships.

In both test facilities, the total flow was measured in a section of closed conduit in the return line using differential pressures produced by area contractions. The bypass flow was regulated by a weir so that the bypass entrance velocity was equal to the approach velocity within the MIS. Water from the bypass flowed over the weir into a separate tank, which was kept at a constant level using a pump with a flow meter in its discharge line. By regulating the pump flow to maintain a constant water level in the tank, the bypass flow and velocity could be calculated.

Head losses were measured using pressure taps placed in the walls and floors of the facilities. Where practical, both the 1:6.6 hydraulic model and the 1:3.3 facility had pressure taps installed in relatively the same locations. Pressure taps were placed in cross-sections upstream of the module entrance, immediately upstream of the screen, at the downstream edge of the screen, downstream of the module, and in the bypass. In each section to be measured, two or four taps were installed and the pressure was physically averaged using a "triple T" method. Each set of pressure taps was compared to a single known water level using a DP cell. Reported losses are from upstream of the intake for either a) the through flow to downstream of the MIS, or b) the bypass flow to the discharge channel just upstream from the control weir.

TEST PROGRAM

An extensive test program was developed to determine all hydraulic characteristics of the MIS system related to efficient fish passage and other practical operational considerations. Specifically, the test program examined the intake shape, velocities at and above the screen face, various bypass geometries, and head losses.

Four possible lateral bar supports (Figure 1) to be supplied with the manufactured wedge wire screen were tested in the small flume to determine their effect on flow patterns through the screen and on head losses. These configurations were tested with 30% and 50% screen porosities, and screen angles of 10, 15, and 20° to the flow. The different configurations tested examined the effect of screen "U clip" spacing and the angle of the additional support bars supplied behind the screen. The slotted clips are used by the manufacturer to maintain the spacing of the wedge wires, which controls the screen porosity. These clips are normally on 2.75 inch (7.0 cm) spacing, Configuration "A". Configuration B had every other clip removed. Configurations C and D also had every other clip removed, but had additional backing bars installed on the remaining clips for increased load bearing capacity. In Configuration C, the backing bars were perpendicular to the screen, whereas, Configuration D had the bars at 40° to the screen to reduce head losses.

The MIS intake shape was evaluated in the 1:6.6 hydraulic model to determine the effects of various approach flow orientation on the velocity distribution approaching the screen, with and without a trash rack. Head losses and velocities in a vertical cross section at the upstream end of the screen were measured. The approach flow was varied from parallel to 45° off the longitudinal MIS axis.

Since head losses would be an important factor in applying the MIS, the losses were examined in all three test facilities. In the small flume, loss measurements were performed on various screen porosities, screen angles, and integral lateral bar supports. Overall system losses were measured in the 1:6.6 and 1:3.3 facilities at prototype velocities of 2, 4, 6, 8, and 10 ft/sec (0.61, 1.22, 1.83, 2.44, and 3.05 m/sec). The 1:6.6 hydraulic model was used to determine head loss coefficients for the entire module, the screen and the rotating support frame, and to the bypass. Head loss measurements in the 1:3.3 facility were used to check data on module and bypass losses obtained in the smaller model. The head loss effects of debris on the screen was also evaluated in the 1:3.3 facility (Winchell, et al. 1993).

Velocities along the screen face were measured over a range of approach velocities. In the 1:6.6 model, the 2-D pitot probe was used to establish velocities in a grid just above (1.65 inches (4.2 cm) prototype) the face of the screen, in a vertical longitudinal grid on the centerline of the module, and in a vertical lateral grid above the upstream edge of the screen. Velocities were only measured in a grid 1.65 inches (4.2 cm) (prototype) above the face of the screen in the 1:3.3 facility to check data from the smaller scale hydraulic model.

RESULTS

The LDA measurements in the small flume to determine how the approach flow is affected by the screen are illustrated in Figure 1 for the case of a 50% porosity screen (with U clips at 5.5 inches (14.0 cm)) at an angle of 15° to the flow.

These data illustrate that the approach velocity is essentially unaffected by the screen. The closest measurement to the screen was made at a distance of 0.06 (1.5 mm) inches above the screen, and, for this point, the horizontal component is only slightly reduced while the vertical component is increased from about 0.28 to 0.34 of the approach velocity. Somewhat greater effects may occur locally just upstream from a U clip which incorporates a manufactured support bar perpendicular to the screen, such as in Configuration C of Figure 1.

A vertical traverse above the screen along the MIS centerline from the 1:6.6 hydraulic model, shown in Figure 2, shows that the horizontal velocity component is fairly constant throughout the system. The numbers in Figure 2 represent local velocities which are non-dimensionalized by dividing by the value by the average approach flow, referred to as normalizing the value. The maximum variation of 10% upstream from the bypass occurred near the leading (upstream) edge of the screen, while the maximum variation in the bypass was 13%. Lateral variations of velocities near the screen were negligible, less than 5%. These data were obtained with an approach flow essentially in-line with the MIS longitudinal axis. Measurements at a vertical lateral cross-section at the leading screen edge showed little variation, about 5%, as the approach flow angle was increased from axial to 30° and 45°, confirming a favorable entrance geometry.

Velocities along the screen centerline, obtained in the 1:3.3 facility, are shown in Figure 3, which gives the speed and direction of the velocity 1.65 inches (4.19 cm) (prototype) above the screen at approach velocities of 4.5 and 8.0 ft/sec (1.37 and 2.44 m/sec) (actual). These data show that no dramatic changes in velocity occur along the face of the entire screen. The calculated components parallel to (V_x) and perpendicular to (V_z) the screen, shown in Figure 4, substantiate this conclusion.

Head loss coefficients obtained in the small flume for the 50% porosity screen at a 15° angle for the four configurations A to D of Figure 1 are 1.17, 1.15, 1.65, and 0.98, respectively. Tests at various velocities showed the coefficients for a given scheme were constant. These data show that the loss coefficients for Configurations B and D were essentially the same, leading to the use of Configuration B in the 1:6.6 hydraulic model and Configuration D in the 1:3.3 facility. The highest loss was produced by Configuration C.

The head loss coefficient for the overall MIS is provided in Figure 5. These data are from the 1:6.6 hydraulic model with the trash rack in place, giving the highest losses. No trash rack was used in the 1:3.3 facility. For a typical design velocity of 6 ft/sec, the overall loss of the MIS would be $1.7 \times 6^2/2g$, or about 0.95 ft (0.3 m). The increase in loss coefficient at the lower velocities show some Reynolds number (viscous) effects, presumably from the smooth MIS surfaces. The loss coefficient for the bypass flow is given in Figure 6 for the four (4) designs investigated. The decrease in losses from Scheme A to Scheme

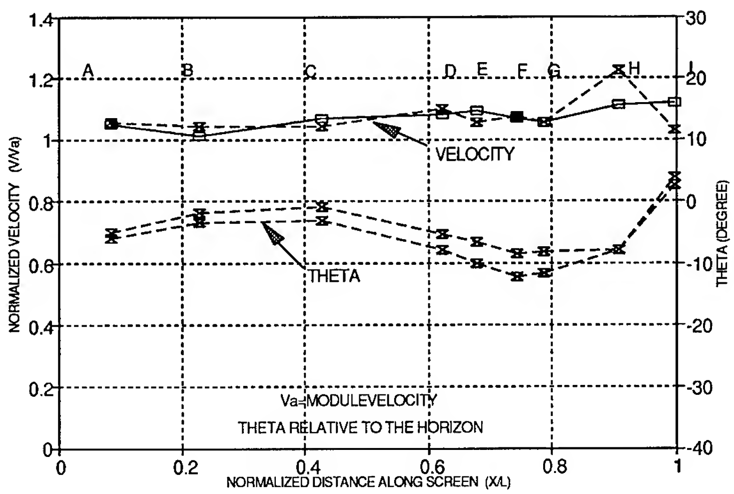


FIGURE 3 AVERAGE LONGITUDINAL VELOCITY PROFILE 1:3.3 FACILITY

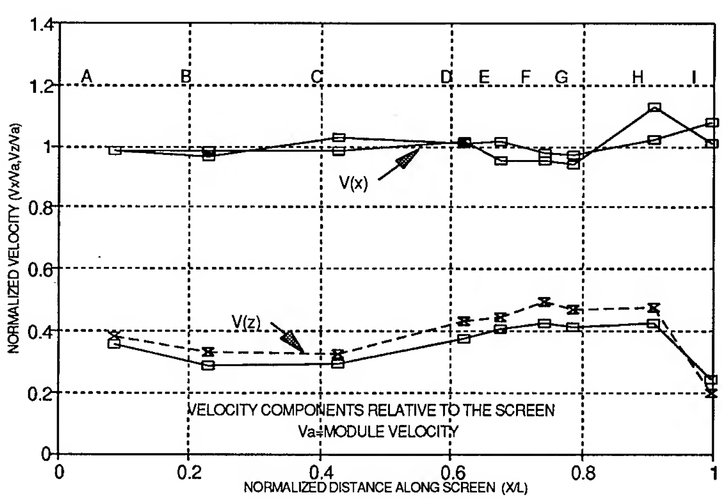


FIGURE 4 AVERAGE LONGITUDINAL VELOCITY COMPONENTS 1:3.3 FACILITY

C is evident. Comparing data of Figures 6 and 5 indicates that the MIS has less loss through the bypass than through the screen.

CONCLUSIONS

The Modular Inclined Screen (MIS) has favorable hydraulic characteristics, and may be used in a variety of approach flow orientation and flow magnitudes conditions with consistent results. With approach velocities ranging from 2 to 10 ft/sec (0.61 to 3.05 m/sec), the normalized velocities along the screen face are nearly constant, and fish moving along the screen would not experience dramatic velocity changes. Finally, the head losses created with the MIS system are relatively small.

ACKNOWLEDGEMENT

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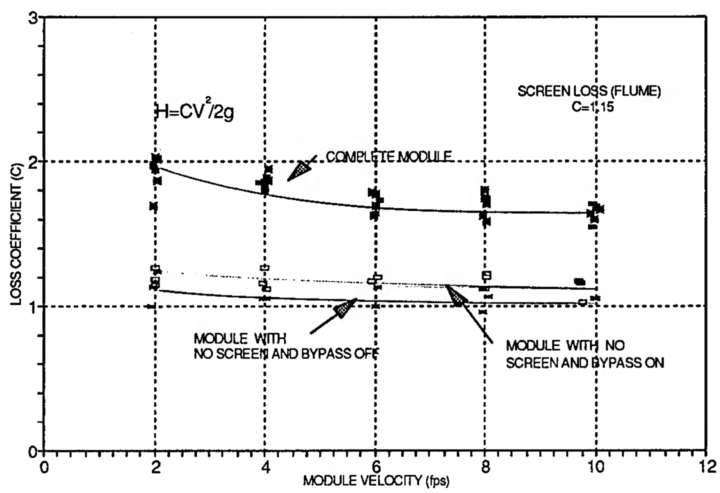

FIGURE 5 THROUGH FLOW HEAD LOSS COEFFICIENT
1:6.6 HYDRAULIC MODEL

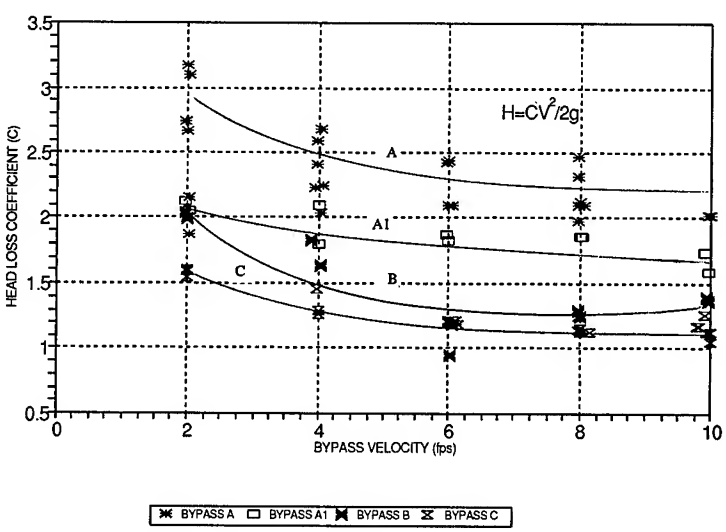

FIGURE 6 BYPASS FLOW HEAD LOSS COEFFICIENT
1:6.6 HYDRAULIC MODEL

Biological Evaluation of a Modular Fish Screen

Fred Winchell[1], Steve Amaral[2], Ned Taft[3], Charles Sullivan[4]

Abstract

The Electric Power Research Institute (EPRI) has developed and is presently testing a new type of fish diversion screen known as the Modular Inclined Screen (MIS). The screen is designed to operate at high water velocities (up to 3.0 ms^{-1}) and is, therefore, significantly more compact than conventional low velocity screening systems. A biological evaluation of the MIS was conducted in 1992 with juveniles of six fish species: bluegill, walleye, rainbow trout, channel catfish, and two alosid species that were tested as one group. The results of this laboratory study demonstrate that the MIS has excellent potential for providing effective fish protection at water intakes.

Introduction

The Electric Power Research Institute (EPRI) has been conducting research since 1985 to develop and evaluate a number of technologies that can be used to prevent fish from passing through hydroelectricturbines. In 1991, EPRI developed a new fish diversion concept which is known as the Modular Inclined Screen (MIS). The modular design is intended to provide flexibility for application at any type of water intake. Installation of multiple units at a specific site can provide fish protection at any flow rate.

[1]Senior Scientist, Stone and Webster Environmental Services, 245 Summer Street, Boston, MA 02210

[2]Scientist, Stone and Webster Environmental Services

[3]Program Manager, Stone and Webster Environmental Services

[4]Program Manager, Electric Power Research Institute

To determine the viability of the MIS as a fish protection system, laboratory studies were conducted in 1992 at the Alden Research Laboratory, Inc. (ARL), to 1) test the design configuration which yields the best hydraulic conditions for safe fish passage, and 2) determine the biological effectiveness of the optimal design in diverting selected fish species to the bypass. The results of the hydraulic evaluation are presented in a separate paper (Cook et al. 1993). The first phase of a two-year biological testing program performed in a laboratory setting was completed in November of 1992; a second phase is planned for the spring of 1993. The results of the 1992 biological evaluation are presented in this paper.

Design Development

The module consists of an entrance with trash racks, dewatering stop log slots, an inclined wedgewire screen set at a shallow angle (10 to 20 degrees) to the flow, and a bypass for directing diverted fish to a transport pipe. The screen comprises 50% porosity Hendrick profile bar with 1.9 mm spacing and is mounted on a pivot shaft so that it can be cleaned via backflushing. The module is completely enclosed and is designed to operate at water velocities ranging from about 0.6 to 3.0 ms^{-1} depending on the species and life stages to be protected. Plan and section views of the MIS are shown in Figure 1.

The MIS design was initially refined during hydraulic studies conducted at ARL with a 1:6.6 model (Cook et al. 1993). The results of hydraulic model tests demonstrated that the MIS entrance design created an acceptable velocity distribution, even when approach flows were skewed as much as 45 degrees. The modular design features were effective in developing uniform velocities over the screen surface without any high velocity zones. Several bypass geometries that appeared to offer the potential for effective and safe fish passage were evaluated. Also, bypass flows were regulated such that the water velocity at the bypass entrance was equal to the module water velocity.

Using the refined design developed during the hydraulic model studies, a 1:3.3 scale model was constructed for biological testing. The biological study was designed to assess fish passage at a screen angle of 15 degrees. Biological tests with additional screen angles may be conducted in 1993.

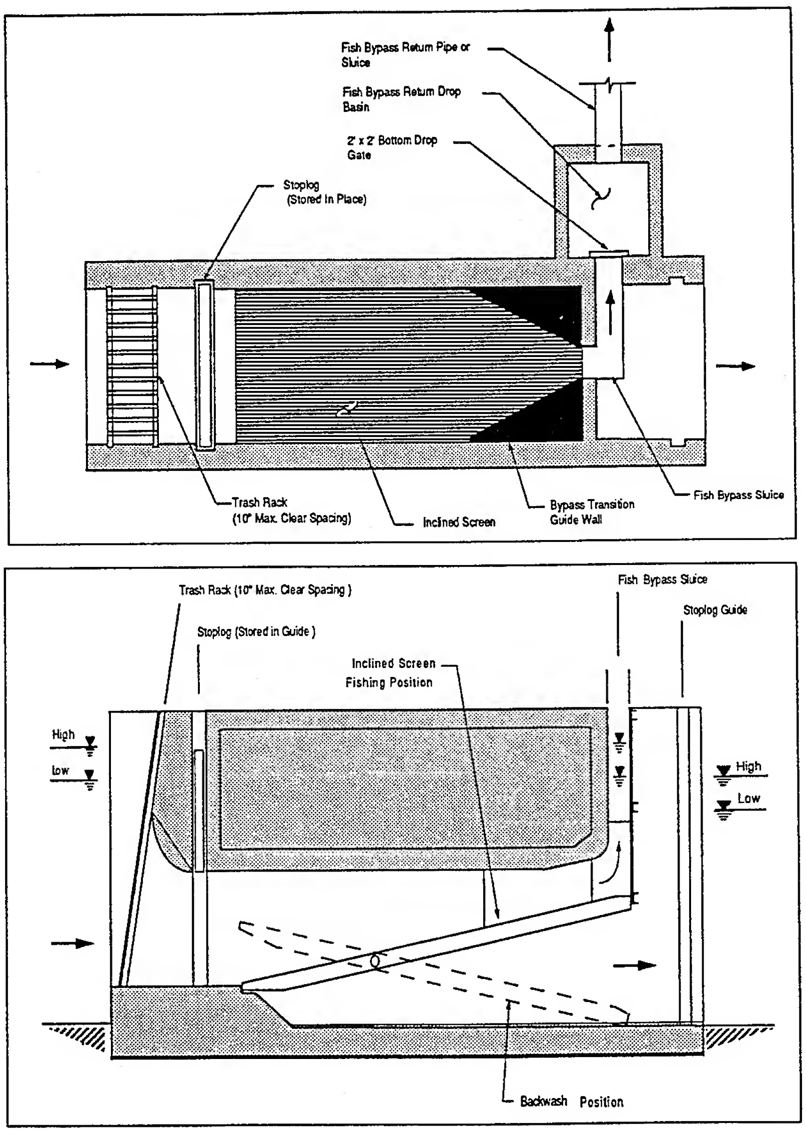

Figure 1. Plan and section view of the Modular Inclined Screen (MIS).

Evaluation Methods

Test species were selected to represent the types of fish of greatest concern at water intakes in the United States, based on a review of turbine entrainment and mortality studies conducted in recent years (EPRI 1992). One size class was tested for each species, except rainbow trout (two size classes were used). Tests were performed with two screen conditions: 1) a clean screen; and 2) a screen with varying degrees of debris accumulation. Rainbow trout fry were the only species used for debris tests. All species were used in clean screen tests. Debris tests were conducted to assess the ability of the screen to be cleaned by backflushing and to determine the effect of debris accumulation on flow conditions and fish passage. Pine needles (Ponderosa and Jeffrey pine), deciduous leaves (white birch, black oak, red maple, and sugar maple), and aquatic vegetation (*Vallisneria* spp.) were used individually in three separate debris test series. Clean screen fish passage tests were conducted at module velocities of 0.6, 1.2, 1.8, 2.4, and 3.0 ms^{-1}; debris tests were performed at the these same velocities with the exceptions of 0.6 and 3.0 ms^{-1} and the addition of 0.4 ms^{-1}.

Debris tests were conducted after incremental amounts of waterlogged debris were added and a target level of head loss above the clean screen condition was achieved. After a fish passage test was performed, debris was added until the next target level was reached. After all target levels had been tested, the screen was rotated and flushed (maintaining the test velocity condition) and the head loss associated with any residual debris remaining on the screen was recorded. A fish passage test also was conducted at each velocity with the residual debris still present on the screen.

Generally, three replicate tests were performed for each species and module velocity for tests with a clean screen. Additional replicates were conducted with bluegill and walleye after a pressurized injection system was installed to introduce test fish into the flume and to assess the effects of handling procedures (i.e., replicates were performed with minimum handling of fish). Also, because alosids are extremely sensitive to handling, one velocity and four replicates were tested per day to allow for optimal latent mortality holding conditions. For pine needle debris tests, one velocity was tested per day and up to seven tests were conducted with increasing amounts of debris and subsequent increases in head loss. Tests with leaves and aquatic vegetation were performed in the same manner as pine needle tests, but only at a water velocity of 1.8 ms^{-1}.

If the impingement rates were high for any species at lower

velocities, then tests at 3.0 ms^{-1} were eliminated for that species. Debris tests were discontinued for any given velocity if high impingement rates were incurred or when a head loss of at least 30.0 cm was achieved and tested. A sample size of approximately 50 test fish and 50 controls was used for all tests conducted with bluegill, rainbow trout fry, and channel catfish; 25 test fish and controls were used for tests conducted with walleye and alosids. Fifty test and control fish were used for the first replicate at each velocity for rainbow trout juvenile tests, but due to a shortage of fish later replicates were reduced to 25 per group and the controls were eliminated from one replicate at 0.6 and 1.2 ms^{-1}.

For the first three replicates performed for each velocity with bluegill and walleye, test fish were released about 9 m upstream from the MIS entrance. However, low recapture rates resulted because fish remained upstream of the screen, particularly at velocities of 0.6 and 1.2 ms^{-1}. For all subsequent tests, a pressurized injection system was used to release fish about 30 cm from the center of the MIS entrance. A net pen was placed below the bypass exit to collect all diverted test fish. Control groups were released directly into the net pen immediately following the release of test fish. After collection from the pen, test and control fish were examined for scale loss or other injuries and held for a minimum of 72 hours to assess latent mortality. Scale loss was noted to the nearest 10% During injury evaluations fork lengths were measured to the nearest millimeter.

After each test was completed the screen was examined to determine the number and location of all fish that were impinged. If possible, impingements were removed from the screen before the next test began. Diversion efficiency was calculated as the ratio of fish collected live to the total number of fish collected (i.e., live and dead fish collected from the net pen and the number of impingements combined).

Results

The average size of the species tested ranged from approximately 48 mm (bluegill and rainbow trout fry) to just under 90 mm (walleye and channel catfish) (Table 1). For clean screen tests, the diversion efficiency of rainbow trout fry and juveniles exceeded 99% at velocities of up to 1.8 and 2.4 ms^{-1}, respectively (Table 2). Channel catfish were diverted at rates exceeding 98% at all velocities including 3.0 ms^{-1}. The diversion efficiency of bluegills and walleye increased during replicates conducted with minimum fish handling; 98% or more at water velocities of up to 1.8 ms^{-1} for walleye and 2.4 ms^{-1} for bluegill.

Table 1. Means and standard deviations of estimated fork lengths for fish used in MIS biological tests.

Species	Group	n	Mean Length (S.D.) (mm)	Range (mm)
Bluegill* (bucket release)	Test	93	47 (5)	37-64
	Control	144	48 (6)	37-68
	Combined	237	48 (6)	37-68
Bluegill (pressurized injection)	Test	104	47 (6)	32-65
	Control	140	47 (6)	34-66
	Combined	244	47 (6)	32-66
Walleye* (bucket release)	Test	54	82 (7)	74-93
	Control	68	84 (5)	74-97
	Combined	122	83 (6)	74-97
Walleye (pressurized injection)	Test	63	85 (4)	77-98
	Control	72	87 (6)	74-100
	Combined	135	86 (5)	74-100
Rainbow Trout Fry (clean screen tests)	Test	84	47 (5)	35-59
	Control	133	49 (5)	37-61
	Combined	217	48 (5)	35-61
Rainbow Trout Fry (debris tests)	Test	149	52 (5)	29-63
	Control	175	51 (5)	35-64
	Combined	324	51 (5)	29-64
Rainbow Trout Juveniles	Test	90	66 (6)	54-83
	Control	102	66 (6)	53-84
	Combined	192	66 (6)	53-84
Channel Catfish	Test	139	88 (12)	52-117
	Control	146	89 (10)	56-118
	Combined	285	88 (11)	52-118
Alosids (blueback herring and American shad)	Test	54	76 (5)	65-87
	Control	72	74 (5)	62-85
	Combined	126	75 (5)	62-87

* Early tests with bluegill and walleye were initiated by releasing test groups into the test flume from a bucket ("bucket release"); low recovery rates led to installation of a pressurized injection system which introduced test fish closer to the module opening and increased recovery rates.

The diversion efficiency of juvenile alosids was lower than that found for other species, except for the 97% alosid diversion rate observed at 1.2 ms^{-1} (Table 2). Alosids also suffered high levels of latent mortality for both test and control fish (Table 3). The high impingement and latent mortality rates experienced by alosids probably resulted from high levels of stress produced by handling at low water temperatures and scale loss. Field collection, transport, and testing of alosids all contributed to extensive scale loss.

Latent mortality was generally low for all other species during the 72-hour holding period (Table 3). After handling was minimized, no mortality was observed for walleye at any velocity, and the mortality of bluegill did not exceed 2%. Latent mortality of rainbow trout fry and juveniles was low and comparable for test and control fish at velocities of up to 2.4 ms^{-1}. No mortality was observed for channel catfish.

For debris tests at module velocities of 1.2, 1.8, and 2.4 ms^{-1}, fish releases were made at debris accumulation levels associated with incremental head losses (loss caused by debris) ranging from about 0.3 cm to 30.5 cm and with residual debris remaining after backflushing of the screen. Debris tests conducted at 0.4 ms^{-1} were limited to incremental head losses of 0.3 and 2.7 cm, which required nearly all of the available debris (the screen was fully covered with 5 to 15 cm of pine needles at a head loss of 2.7 cm).

Rotation of the screen for brief periods (approximately 1 minute) flushed the large majority of debris from the screen at all velocities. The head loss associated with this residual debris (above that measured for a clean screen) ranged from 0.3 cm at 0.4 cm^{-1} to 5.5 cm at 2.4 ms^{-1}. Tests conducted with other debris types at a module velocity of 1.8 ms^{-1} indicated that fresh-fallen leaves and aquatic vegetation produced greater levels of head loss than pine needles but flushed more completely from the screen. After backflushing, residual head loss associated with leaves was not detectable (<0.3 cm) and was estimated at approximately 0.3 cm for aquatic vegetation.

During pine needle tests with module velocities of 0.4, 1.2, and 1.8 ms^{-1}, fish impingement did not occur at incremental head losses of less than 7.5 cm. Some impingements occurred at greater levels of head loss, but diversion efficiency generally remained above 90% at these velocities. At 2.4 ms^{-1}, diversion efficiency was comparable to that observed for a clean screen (about 95%) for head loss levels up to 1.5 cm, but diminished to about 73% at head losses of 15.0 and 30.0 cm. Injury to test fish was minimal and generally comparable to controls, averaging less than 1% at all four velocities tested. Latent mortality was

Table 2. MIS diversion data summary for all tests and species, except debris tests with rainbow trout fry.

Species	Test Flume Temperature Range (degrees Farenheit)	Percent Diverted Live 0.6 m/s (2 fps)	1.2 m/s (4 fps)	1.8 m/s (6 fps)	2.4 m/s (8 fps)	3.0 m/s (10 fps)
bluegill (bucket release)	61 - 71	100.0	100.0	94.1	89.5	79.7
bluegill (pressurized injection)	55 - 59	97.6	99.2	100.0	97.8	---
bluegill (minimum handling)	46	100.0	99.0	99.0	98.0	96.2
walleye (bucket release)	61 - 71	100.0	100.0	93.0	90.8	86.5
walleye (pressurized injection)	55 - 59	100.0	100.0	98.5	91.7	---
walleye (minimum handling)	46	100.0	100.0	99.0	95.2	100.0
rainbow trout fry	51 - 56	98.5	100.0	100.0	95.2	95.6
rainbow trout juveniles	41 - 46	100.0	100.0	100.0	99.1	96.3
channel catfish	46	99.2	100.0	100.0	99.4	98.6
alosids juveniles* (blueback herring and American shad)	46 - 47	89.1	97.0	81.6	63.5	---

* Results may have been influenced by collection and transport stress and because fish were collected from the tail-end of the outmigration at lower temperatures.

Table 3. MIS latent mortality data summary for all tests and species (except debris tests conducted with rainbow trout fry).

	72 hr Latent Mortality (%)									
	0.6 m/s (2 fps)		1.2 m/s (4 fps)		1.8 m/s (6 fps)		2.4 m/s (8 fps)		3.0 m/s (10 fps)	
Species	Test	Control	Test	Control	Test	Control	Test	Control	Test	Control
bluegill (bucket release)	0.0	18.2	23.7	23.4	16.2	16.1	27.9	18.4	40.8	14.5
bluegill (pressurized injection)	21.3	10.5	14.5	17.0	18.8	10.5	31.6	14.2	---	---
bluegill (minimum handling)	0.0	---	0.0	---	2.0	---	0.0	---	2.0	---
walleye (bucket release)	100.0	7.7	6.3	1.4	0.0	3.8	4.3	0.0	9.6	10.8
walleye (pressurized injection)	11.4	7.4	21.0	11.6	14.9	2.7	10.6	9.7	---	---
walleye (minimum handling)	0.0	---	0.0	---	0.0	---	0.0	---	0.0	---
rainbow trout fry	7.5	1.6	0.0	0.0	0.0	3.3	2.5	2.6	4.6	0.8
rainbow trout juveniles	0.9	2.0	0.8	0.0	0.0	1.1	1.9	1.7	7.6	1.0
channel catfish	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
alosids juveniles**	63.2	58.1	39.8	36.4	38.8	32.7	86.9	88.8	---	---

* This latent mortality estimate is based on only one fish.

** Results may have been influenced by collection and transport stress and because fish were collected from the tail-end of the outmigration at lower temperatures.

generally low and comparable for test and control groups at module velocities of 0.4, 1.2, and 1.8 ms^{-1}, even for tests conducted with high levels of debris blockage.

Debris tests with leaves and aquatic vegetation showed that the relationship between head loss and fish impingement was similar for all debris types tested. No impingements occurred at 3.0 cm of head loss and only minor impingement (<5%) at 7.5 cm. At greater levels of head loss, impingement rates were higher for tests with leaves and aquatic vegetation than tests with pine needles. Only one fish was injured out of the 137 test fish recovered during leaf and aquatic vegetation tests. No latent mortality was observed during the 72 hour holding period.

Conclusion

The results of the biological testing clearly demonstrate that the MIS can effectively and safely divert fish to a bypass. Impingement and latent mortality is generally low, even at approach velocities as high as 3.0 ms^{-1}. Also, it was determined that debris can be effectively removed by rotating the screen for backflushing and that minor levels of debris accumulation have little effect on fish passage. The biological effectiveness that has been demonstrated to date combined with the modular design of the screen system, supports the conclusion that the MIS is a cost-effective, viable fish protection technology.

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Review of Fish Entrainment and Mortality Studies

Ned Taft[1], Fred Winchell[2], John Downing[3],
Jack Mattice[4] and Charles Sullivan[5]

Abstract

The potential effects of fish passage through turbines at hydroelectric projects has been the subject of extensive research over many decades. Until recently, efforts were concentrated on determining mortality rates of anadromous species, primarily salmonids. The ongoing FERC relicensing process in the United States, coupled with extensive fish restoration efforts on many river and lake systems, has led to intensified efforts to define passage and mortality rates of resident, as well as migratory, fish species. This paper summarizes a review funded by the Electric Power Research Institute which compiled much of the work recently conducted in this field.

Introduction

Entrainment and turbine mortality are two of the most significant issues currently faced by applicants and resource agencies during the licensing and relicensing of hydroelectric facilities. Until recently, little information was available on the rates at which fish (particularly non-migratory "resident" species) are entrained, and the mortality caused during turbine

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[1] Program Manager, Stone & Webster Environmental Services, 245 Summer Street, Boston, Massachusetts 02210
[2] Senior Scientist, Stone & Webster Environmental Services
[3] Principal Scientist, Stone & Webster Environmental Services
[4] Program Manager, Electric Power Research Institute, 3412 Hillview Avenue, Palo Alto, California 94304
[5] Program Manager, Electric Power Research Institute

passage was understood relatively well only for juvenile salmonids. Many of the hydro projects undergoing relicensing at this time are being requested to address these issues for resident species of fish and/or migratory clupeid species (shad, alewives and herring).

The limited information available on entrainment and turbine mortality has made it difficult to assess fisheries impacts at many hydro projects. As a result of the FERC relicensing process, many studies addressing these issues were initiated in the past several years. Recognizing the need to disseminate the growing base of data available on passage and mortality rates to others confronting the turbine passage issue, the Electric Power Research Institute (EPRI) funded a review of recent studies and results. This paper presents a summary of the review, which is presented in full in EPRI Report TR-101231 entitled "Fish Entrainment and Mortality Review and Guidelines" published in September 1992.

Methods used to study entrainment

Three primary methods for evaluating fish entrainment were identified: netting, hydroacoustics and telemetry techniques. Tailrace netting was the method most commonly employed and was frequently combined with studies of turbine mortality. Most studies used a net which sampled the entire discharge from one or more operating units. In several studies, nets were deployed at other locations (at the power canal entrance or turbine intake), while in other studies only part of the outflow from a turbine was sampled. When tailrace conditions are conducive to sampling and the turbine output is not excessive, a full-flow tailrace net is preferred to ensure that a representative sample is collected. Netting enables the size and species composition of entrained fish to be evaluated; however, many studies suffer from low catch efficiencies for smaller-sized fish. In order to retain fish smaller than 100 mm in length, it was found that the net mesh should have a square mesh no larger than 0.5 inches (12 mm).

Entrainment studies using hydroacoustic techniques have usually employed periodic netting to assess species composition and to verify fish counts. Although some studies have shown relatively good correspondence between net and hydroacoustic counts, other studies have had less success. Most studies have used the hydroacoustic data to compare entrainment rates between units and to assess the rate of entrainment in the intervals between net sampling periods. Radio telemetry techniques have been most effectively used to follow the movements of migratory species in order to evaluate exposure to turbine

passage, the effectiveness of fish bypasses, delays in passing dams, and for siting of fish passage facilities.

Methods used to study turbine mortality

The primary methods used to evaluate turbine mortality include tailrace netting, balloon tagging and radio telemetry techniques. Tailrace nets offer the advantage of allowing mortality to be evaluated for naturally-entrained fish, eliminating the stresses associated with transport, handling and introduction when using introduced fish. However, many studies employing tailrace nets were affected by net-related mortality, which often compromised the accuracy of turbine mortality estimates. By evaluating the characteristics of the nets used in these studies, it was found that high rates of net mortality could usually be ascribed to injuries caused in the cod end of the net (when no live car was used to provide a low-velocity refuge for collected fish), the use of knotted net material, or the use of a net with a short taper (less than 3:1 length-to-width ratio).

The balloon tag method involves attachment of a self-inflating tag to the fish, introducing them into the turbine intake, and recovering the fish when the inflated balloon buoys the fish to the surface of the tailrace. This method has been shown to be effective for evaluating turbine mortality rates, particularly for species which are typically stressed or injured when recovered in a collection net. The method usually achieves a relatively high rate of fish recovery, which can produce credible estimates of turbine mortality with relatively small sample sizes. The primary disadvantage of the method is the large amount of labor involved in fish recovery, which could be prohibitive in studies at a given site where multiple species and size classes of fish and/or multiple turbine operating conditions must be evaluated. Radio tags are sometimes used in conjunction with balloon tags to improve fish recapture rates.

Radio tags have been used in several studies to assess turbine mortality based on observations of fish movement following turbine passage. Tag regurgitation (if orally-inserted) and loss of signals from fish have limited the accuracy of some studies. Another limitation of this method is that information on turbine injuries and delayed mortality cannot typically be collected.

Results of entrainment studies

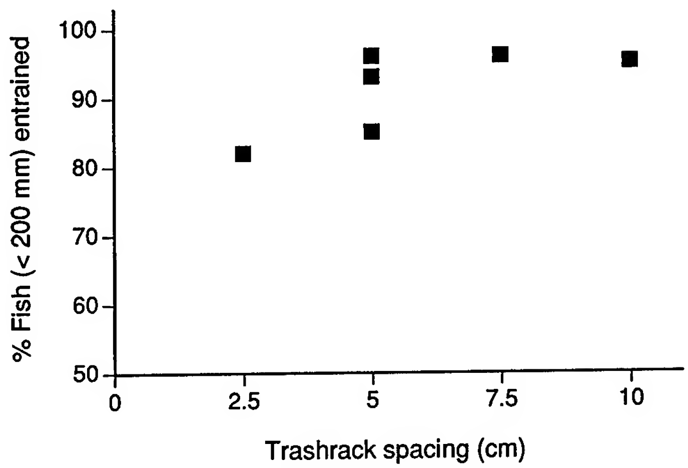
Most of the entrainment studies reviewed were conducted in the states of Michigan and Wisconsin, but the overall trends observed were consistent with observations made in studies conducted in other regions. For those studies which provided size distribution information, small fish (<200 mm) generally comprised over 90% of the fish entrained (Figure 1). Entrainment was usually highest in the late spring and summer and was consistently low in the winter and early spring. Diurnal patterns were variable from site to site, by season and between species. Entrainment rates typically averaged between 1 and 10 fish per hour for each turbine unit sampled (Figure 2).

Results of Turbine Mortality Studies

Researchers that were successful in minimizing net-related mortality were able to demonstrate that the mortality of naturally-entrained mixed resident fish was sometimes as low as 1-2% and averaged approximately 6% at both Francis and Kaplan turbines (Table 1). Higher mortality estimates generally resulted from the studies which used introduced fish. In many cases, increased mortality was probably related to the use of fish larger than the majority of naturally entrained fish. The stress of transport, marking and introduction techniques also likely contributed to mortality in most studies using introduced fish. The mortality estimates for introduced resident species generally fell within the range of 10-30%.

The mortality rates of juvenile clupeid species appears to be much lower than had been indicated in earlier studies using the same species. Recent studies of juvenile American shad and blueback herring using the balloon tag technique indicate an average turbine mortality of approximately 3.6% for Kaplan turbines and 16% for Francis turbines.

The limited information available for other types of turbine runners indicate that bulb turbines may offer slightly lower mortality rates than Kaplan turbines, and that Ossberger Cross-Flow turbines are likely to cause high rates of mortality to larger fish due to the narrow spacing of the runner vanes. Information on the one operating STRAFLO turbine evaluated suggests that the mortality of adult American shad was approximately 20%, while the mortality rate of juvenile clupeids at this site is still the subject of considerable debate. The single study conducted at a tube turbine showed mortality rates ranging from 9-25% for the species evaluated.



Figrure 1. Entrainment of fish less than 200 mm in length for four trashrack spacings.

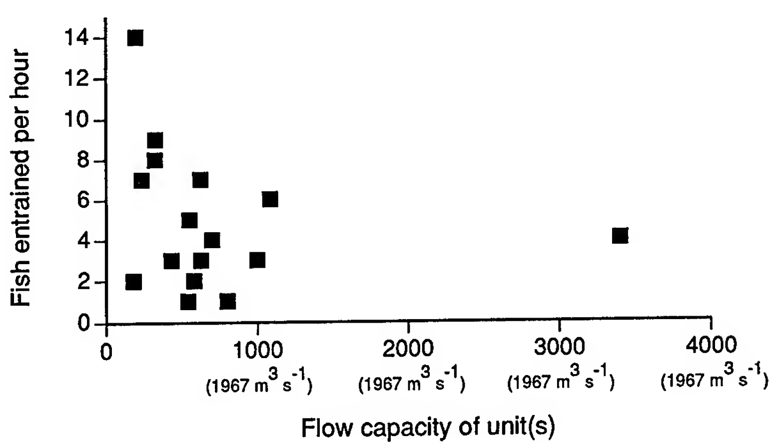


Figure 2. Fish entrainment rates for hydropower turbines sampled with full-flow tailrace nets.

Table 1. Average Mortality Rates by Species Group and Turbine Type

Species Group	Francis Turbines			Kaplan Turbines		
	Sites[1]	N[2]	Avg.	Sites[1]	N[2]	Avg.
Introduced Fish						
Salmonids	4	8	18.2%	2	2	7.6%
Clupeid juveniles	3	3	16.0%	4	6	3.6%
Clupeid adults	1	1	28.6%	2	2	19.1%
Centrarchids	6	17	11.7%	2	3	8.5%
Percids	4	10	23.6%	0	0	na
Esocids	4	5	22.3%	0	0	na
Catostomids	5	10	24.0%	0	0	na
Cyprinids	5	11	20.0%	0	0	na
Ictalurids	0	0	na	1	4	11.3%
Mixed resident sp.	1	1	37.0%	2	2	30.2%
Entrained Fish						
Mixed resident sp.	4	4	5.8%	2	2	6.3%
AVERAGE	--	--	20.7%	--	--	12.4%

[1] Number of individual turbines tested (more than one at some sites).
[2] Number of observations including tests conducted with different size groups or at different operating conditions.

Research Update on the Eicher Screen at Elwha Dam

Fred Winchell[1], Ned Taft[2], Tom Cook[3] and Charles Sullivan[4]

Abstract

The Electric Power Research Institute (EPRI) has conducted two years of biological evaluations of an Eicher Screen installed in a 9-foot (2.7 m) diameter penstock at the Elwha Hydroelectric Project in Washington state. Testing has shown passage survival to equal or exceed 98.7% for steelhead smolts, coho smolts, chinook smolts, coho fingerlings and chinook fingerlings. Scale loss injuries were minimal at velocities of 4-6 fps, but increased at higher velocities. Most injuries occurred from fish contacting the screen in an area where it transitions from 63% porosity to 32% porosity wedgewire material, where the velocity component perpendicular to the screen was relatively high. Hydraulic model studies conducted in 1992 indicated that a more gradual reduction of the porosity in the downstream end of the screen would reduce the observed velocity peak by approximately 10%, but would not achieve a fully uniform flow distribution.

Introduction

The Eicher Screen prototype was installed by James River II, Inc. in a 9-foot diameter penstock to one of the four 3.2 MW units at the Elwha Hydroelectric Project. The elliptical screen is sloped upwards at a 16 degree angle and serves to guide fish towards a bypass at the top of the penstock (Figure 1).

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[1] Senior Scientist, Stone & Webster Environmental Services, 245 Summer Street, Boston, Massachusetts 02210
[2] Program Manager, Stone & Webster Environmental Services
[3] Senior Hydraulic Engineer, Stone & Webster Environmental Services
[4] Program Manager, Electric Power Research Institute, 3412 Hillview Avenue, Palo Alto, California 94304

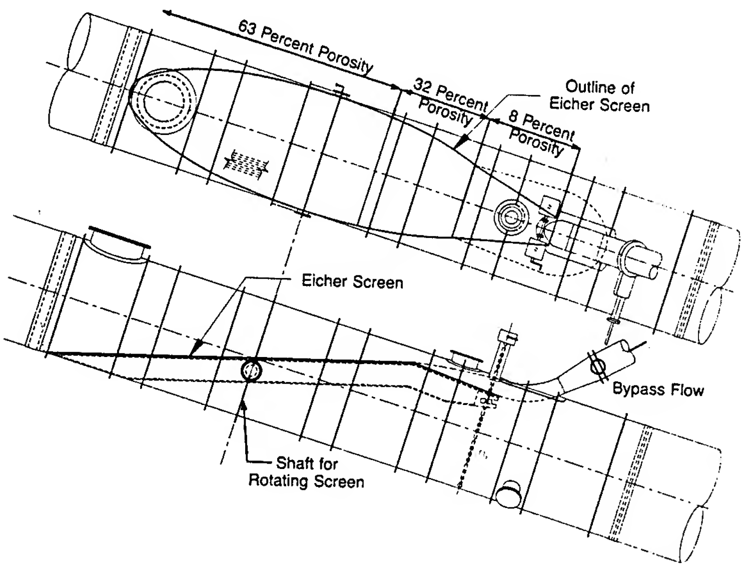

Figure 1. Plan and view sections of the Elwha Eicher Screen.

The screen is composed of panels of profile bar (commonly referred to as wedgewire) material, with reduced porosity towards the downstream end of the screen. The screen design was developed based on fish passage information gained during laboratory studies conducted by EPRI in 1984 and 1985 and a hydraulic model study conducted by James River II, Inc. in 1989. The design was initially developed for application at Portland General Electric's T.W. Sullivan plant, and is commonly referred to as an "Eicher Screen," after its inventor.

Testing performed in 1990 demonstrated the screen's effectiveness in diverting coho salmon smolts. Papers presented at Waterpower '91 reported the history of EPRI's involvement in development of the Eicher Screen design, factors which influenced the design used at Elwha, and results of the 1990 biological tests. The current paper summarizes the results of testing performed in both 1990 and 1991 with seven size classes and three species of salmonids, the results of hydraulic model studies performed in 1992 to evaluate refinements in the screen's design, and implications for the design of future installations.

Prototype Design

James River II Inc. contracted with Hosey and Associates Engineering Company to oversee hydraulic model studies and design the prototype screen. Hosey, in turn, contracted with Engineering Hydraulics, Inc., to build the model and conduct the laboratory tests. Two major refinements were made to the screen design during the hydraulic model studies. The design of the support structure was streamlined in order to reduce headloss, and the porosity (percent open area) of the screen was reduced in the downstream end of the screen to provide a more uniform flow field over its entire length.

The prototype with the refined design was installed in the spring of 1990 as part of a 46.5-ft (14 m) long, prefabricated penstock section. The screen is located 15 feet (4.5 m) downstream of a 15-degree bend in the penstock (Figure 1). The inclined portion of the screen is comprised of two sections with uniform bar width (0.073-inch or 1.9 mm) but different bar spacing. The upstream section is 20-feet in length, has a porosity of 63 percent with an opening between bars of 0.125-inches (3.2 mm). The downstream section is 7.5-feet in length and has a screen porosity of 32 percent with an opening between bars of 0.035-inches (0.9 mm). The section of screen in the bypass transition is 7 feet in length and has a porosity of 8 percent, with an 0.093-inch (2.4 mm) bar width and an 0.008-inch (0.2 mm) opening between bars. The entire screen including the transition section is designed to pivot so that it can be cleaned by backflushing or put into a position parallel to the penstock when not in use.

Test Facilities

Stone & Webster Environmental Services was retained by EPRI to design the evaluation facilities (Figure 2) and to oversee the testing program. Two pressurized release systems were used to release test fish into the intake and penstock upstream of the screen, and a third system was used to release control fish into the collection tank. Each system was composed of a fish release tank connected to an 8-inch (0.2 m) diameter release pipe. The fish were released by gradually displacing the water and fish using compressed air. The penstock system released fish into the base of the penstock approximately 25 feet (7.6 m) upstream of the leading edge of the screen. The intake release system was positioned approximately 20 feet (6 m) further upstream.

Bypassed fish were delivered into the collection tank through a 24-inch (0.6 m) pipe, which discharged the bypass flow and fish upward

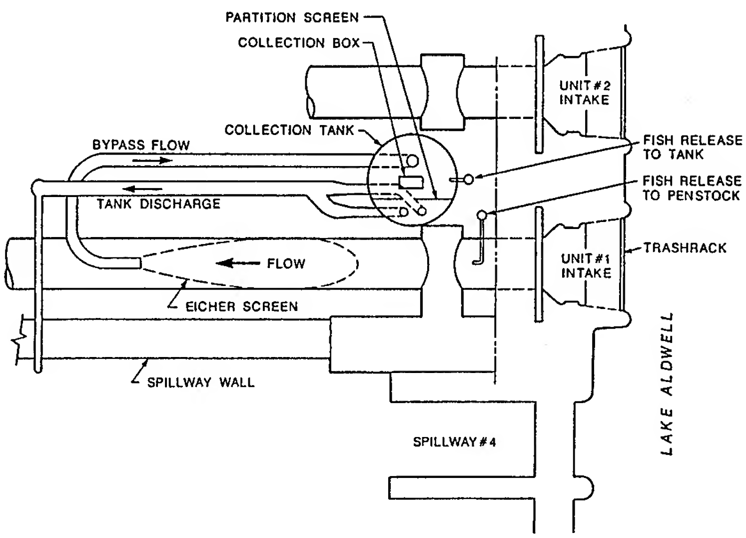

Figure 2. Plan view of the Elwha evaluation facilities.

vertically through an open sluice gate at the floor of the tank. Bypass flows were regulated by adjusting the elevation of the water in the tank. The water level in the tank was controlled by adjusting two 20-inch (0.5 m) valves which drained flow from the tank behind a screen partition designed to retain collected fish in the tank. When a test was completed, the tank was drained and the fish were guided by the sloped floor into a collection box.

Test Methods

Screen effectiveness was evaluated by determining the proportion of fish diverted live, the proportion of fish that were injured during passage by the screen, and survival four days after screen passage. Penstock velocities ranging from 4 to 7.8 fps and bypass velocities from 4 to 8.6 fps were evaluated. Each test combination of penstock and bypass velocities was replicated several times in daytime and at night. Limited test series were also performed to evaluate the effect of penstock lighting, introduced debris and lower bypass flows on fish passage.

Test fish were obtained from state and tribal hatchery facilities, using fish stocks that are either indigenous or are routinely planted in the Elwha River. Each species was tested in its migratory (smolted) lifestage, as well in the smaller fry and/or juvenile lifestages. ATPase activity of the smolts was monitored to assure that the fish were tested in peak migratory condition.

Before testing, fish were marked with one of four colors of dye pneumatically injected at one of seven locations, producing a total of 28 distinct marks. Marked groups of 100 fish each were held in square fiberglass tanks situated on the middle deck of the evaluation facility. Each fish was later examined to assure that its mark was visible, to cull out any fish with significant scale loss or other injuries, and to obtain an accurate count of the fish remaining in each mark group.

At the initiation of testing each day, the Eicher Screen was moved from the neutral position (with the screen parallel to the penstock flow) to the fishing position (with the screen at a 16 degree angle to the penstock). Penstock and bypass flows were then set to the first scheduled test condition. A final count was then made as the fish were transferred into buckets. Next, the fish were poured into the appropriate release tanks and the covers were closed and sealed. The fish were then gradually purged from the release systems.

The bypass flow specified for the test condition was maintained for five to ten minutes after fish were released. When the bypass velocity was 7 fps or less, a run time of 10 minutes was used. At a 7.8 fps bypass velocity, the run time was reduced to 5 minutes. These durations were found to be sufficient to allow the fish to pass through the system into the collection tank.

After a test was completed, the inlet sluice gate was closed and the collection tank was gradually drained. The collection box was then hoisted to the upper deck of the evaluation facility. Fish were evaluated immediately after recovery, directly from the collection box. Each fish was anesthetized and its dye mark, fork length and condition was recorded. A classification system developed by the National Marine Fisheries Service for studies on the Columbia River was used to categorize injuries. The major categories used were:

- o "partial descaling" (scattered or patchy loss 3 to 16% per side);
- o "descaled" (over 16% scale loss on one side); and
- o "other injuries" (bruises and eye injuries).

All test fish were held in fresh water for four days following recovery to assess delayed mortality.

Test Results

Two years of testing were performed. In 1990, tests were performed with coho smolts and coho fingerling pre-smolts. In 1991, testing was conducted with steelhead smolts, coho smolts, chinook smolts, chinook fingerling pre-smolts, steelhead fry and coho fry.

Detailed results of the 1990 test program are available in EPRI Report No. GS/EN-7036. Comprehensive results for both years, and velocity profiles measured in the prototype, are presented in EPRI Report No. TR-101704.

Test results showed that passage survival (diversion efficiency adjusted for 96-hour survival) equalled or exceeded 98.7% for all three species of smolts tested (Table 1). The results obtained for coho smolts in 1991 were nearly identical to those obtained in 1990. Although the facility was not specifically designed to pass fish smaller than smolts, tests showed that passage survival averaged 99.2% for coho fingerling pre-smolts (average length: 4.0 inches or 102 mm), 99.9% for chinook fingerling pre-smolts (average length: 2.9 inches or 73 mm), 97.1% for steelhead fry (average length: 2.0 inches or 52 mm) and 91.6% for coho fry (average length: 1.7 inches or 44 mm). Excluding tests conducted at penstock velocities of 7 fps or higher, the passage survival of coho fry was 95.9%.

Table 1. Net Passage Survival for the Elwha Eicher Screen.

Species/ Size Class	Average Length (mm)	Average Diversion Efficiency	Adjusted 4-Day Mortality	Net Passage Survival
Steelhead smolts	174 mm	99.6 %	0.2%	99.4%
Coho smolts 1990	135 mm	99.5%	0.1%	99.4%
Coho smolts 1991	145 mm	98.7%	0.0%	98.7%
Coho juveniles	102 mm	99.4%	0.2%	99.2%
Chinook smolts	99 mm	99.7%	0.9%	98.8%
Chinook juveniles	73 mm	99.9%	0.0%	99.9%
Steelhead fry	52 mm	98.2%	1.1%	97.1%
Coho fry All data	44 mm	96.1%	4.7%	91.6%
Coho fry Tests <7 fps		98.0%	2.1%	95.9%

Injuries were generally rare in tests conducted at penstock velocities of 7 fps or less. For all species and lifestages tested except chinook smolts, the proportion of fish with > 16% scale loss on one side ("descaled" as defined in criteria used on the Columbia River) averaged less than 1% at velocities of 4 and 6 fps, less than 2% at 7 fps, and less than 6% at 7.8 fps. Descaling was most common on chinook smolts, which averaged 0.4% at 4 fps, 2.8% at 6 fps, 6.7% at 7 fps and 12.6% at 7.8 fps.

Visual observations indicated that most or all of the injuries were due to fish striking the screen in the vicinity of the transition between the 63% and 32% porosity sections. Velocity measurements taken in 1990 showed that the normal velocity component (perpendicular to the screen) in this area was relatively high compared to upstream areas, averaging nearly half of the penstock velocity. Since fish injury increased at penstock velocities greater than about 7 fps, it appears that a normal velocity component of 3 to 3.5 fps may represent a critical threshold for scale loss for the species tested.

Mortality rates associated with different levels of scale loss varied substantially between species over the 96-hour holding period. Some elevation in mortality was noted for steelhead (15.3% mortality) and chinook smolts (2.8% mortality) with as little as 3-10% scale loss on one side, while coho smolts did not show an appreciable increase in mortality (less than 1 percent) unless they were at least 30-40% descaled. The amount of scale loss required to cause mortality in over half of the fish was in the 20 to 30% range for steelhead smolts, between 30 and 40% scale loss for chinook smolts, and over 50% scale loss for coho smolts.

Injury rates increased substantially when the screen was partially clogged with introduced debris. Debris accumulations that produced over one or two tenths of a foot (25-50 mm) of head loss resulted in a noticeable increase in injury at the higher penstock velocities, particularly for chinook smolts (Figure 3). However, the screen was readily cleaned by rotating it approximately 8 degrees. In over 60 days of testing performed at Elwha, natural debris accumulation over the 8 to 12 hour test periods never caused a noticeable increase in injury or necessitated cleaning of the screen.

No difference in fish injury or diversion efficiency was found in the range of bypass flows evaluated in the primary test conditions. A limited series of tests were conducted with coho and chinook smolts to evaluate the effect of lower bypass flows. The bypass flows evaluated in these tests produced an average velocity at the bypass entrance that was

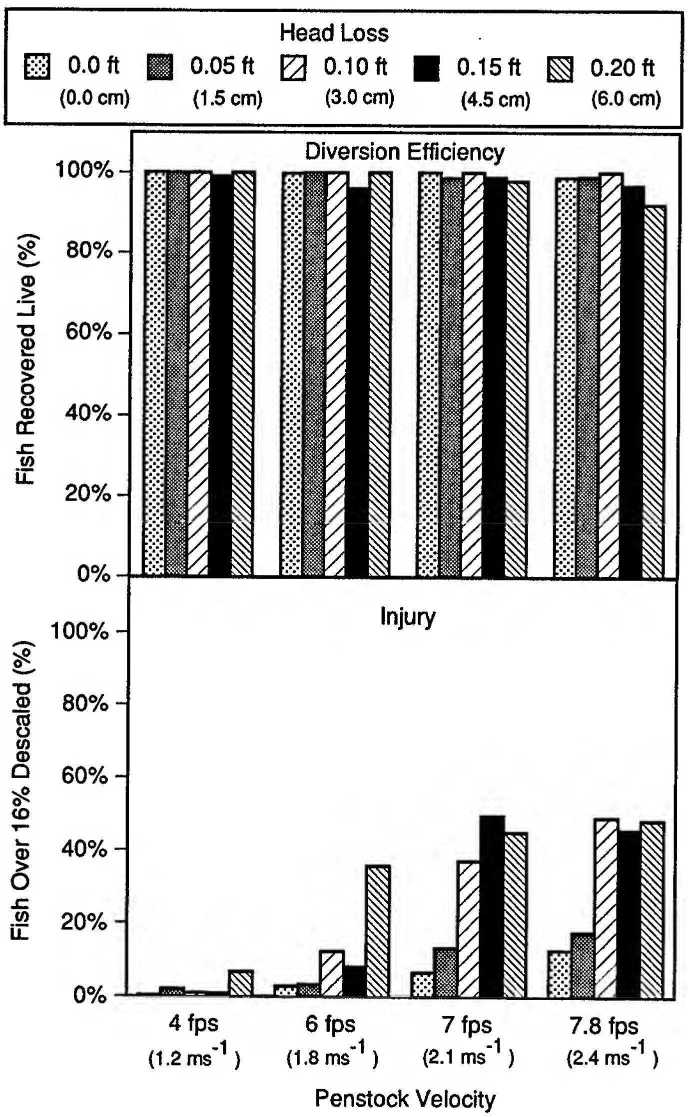

Figure 3. Diversion efficiency and injury of chinook smolts at four penstock velocities and five levels of debris-induced head loss.

90%, 80% and 70% of the average penstock velocity. A minor decrease in bypass efficiency appeared to occur when the velocity at the bypass entrance was less than 90% of the penstock velocity. This equates to a bypass flow that was 4.2% of the turbine flow.

No operational problems were evident during testing, and head loss with a clean screen ranged from 0.5 feet (0.15 m) at 4 fps to 1.9 feet (0.58 m) at 7.8 fps.

1992 Hydraulic Model Studies

EPRI funded a series of hydraulic model tests during 1992 to evaluate the applicability of hydraulic data from Elwha to other sites and to evaluate the potential for further improvement of the flow distribution via porosity control. A 1:4.5 scale model of the Elwha intake, penstock and Eicher Screen was constructed for the test program at the Alden Research Laboratory, Inc. (ARL) in Holden, Massachusetts.

The first series of velocity measurements were made upstream of the screen to assess whether the hydraulic conditions observed at Elwha were influenced by the 15-degree bend in the penstock which occurs about 15 feet upstream of the Eicher Screen. The results showed that the bend had no significant effect on the flow distribution measured at the upstream end of the screen.

In the second series of tests, the potential for creating a more uniform velocity distribution over the length of the screen by providing a more gradual reduction in porosity was evaluated. Specifically, it was hoped that the injury of fish observed at the 63 to 32% porosity transition at Elwha could be eliminated. The downstream half of the 63% porosity section was replaced with two equal length screen sections of 50% and 40% porosity. This resulted in a screen porosity gradation of 63%/50%/40%/32%, compared to the original configuration of 63%/32%. A third configuration with a uniform porosity of 50% was also evaluated. The results indicated that the more graduated porosity configuration yielded the most uniform flow distribution, although the maximum velocity normal to the screen was only reduced by about 10% from the configuration used at Elwha. As expected, the uniform 50% porosity model showed a gradual increase in the normal velocity over the length of the screen which would likely increase the potential for fish impingement and injury.

Conclusions

The results obtained at Elwha indicate that the Eicher Screen can

divert juvenile and smolt-sized salmonids with minimal injury at penstock velocities of up to 7 fps. Descaling injuries, particularly for salmon and steelhead smolts, could result in mortality as velocities increase beyond this range. The results of the hydraulic model studies suggest that the velocity could be increased by approximately 10% if a more graduated porosity configuration were used.

The relatively limited test series conducted with fry-sized fish indicate that passage survival rates exceeding 95% can be achieved for fish in the 1.5-2.0 inch (40-50) mm range at velocities of up to 7 fps. If smaller fish need to be protected, lower design velocities and closer bar spacings on the screen material should be considered.

Testing conducted at different bypass flows indicate that the velocity approaching the bypass should be a minimum of 90% of the penstock velocity. Impingement could become a problem if bypass velocities are substantially lower. Bypass velocities higher than the penstock velocity did not show an appreciable effect on the test results.

The increased injury observed during debris tests suggest that future installations should include monitoring equipment capable of detecting incremental head losses as low as 0.1 to 0.2 feet (2.5 to 5.0 cm). For sites where debris accumulation is expected to coincide with periods of fish movement, a more rapid screen operator would minimize fish losses past the screen. A hydraulic operator might provide the most reliable means for flushing the screen quickly.

Limnological Considerations For Aeration at Mainstem Projects

Richard J. Ruane (AM),[1] Charles E. Bohac (M),[1] and David J. Bruggink[2]

Introduction

Six TVA mainstem hydropower projects on the Tennessee River were studied to determine the most cost-effective approach to increase dissolved oxygen (DO) in the turbine discharges. The annual minimum DO at these mainstem projects is usually greater than 5 mg/L, but in some years, the minimum DO can be less than 3 mg/L. Even though these low DO characteristics are higher compared to the discharges from many TVA storage projects, the cost of aeration at mainstem projects can cost considerably more due to the higher river flows through mainstem projects (thus requiring larger aeration systems). But a special study in 1986 revealed another potential alternative for mainstem projects.

During a drought in 1986, a special study was conducted at Watts Bar Hydropower Project to see if operations at the plant could be modified to increase DO levels in the turbine discharges to alleviate impacts to a mussel bed downstream from the dam. During this investigation it was determined that DO increased about 2 mg/L over a six-hour period after all five units had been placed in service. Also, the DO in the releases from the unit nearest the river bank was about 1 mg/L greater than the DO in the releases from the unit farthest from the river bank. Although data were not collected from the reservoir, these changes in DO in the releases were attributed to changes in the withdrawal zone of water from the reservoir, i.e., as the rate of discharge through the turbines increased, the withdrawal zone expanded in elevation to include more water from the upper portion of the water column where the DO is much higher in concentration. The higher DO observed in unit 1 was attributed to the influence of the bank on the upstream side of the dam on the withdrawal zone, causing even more water having higher DO from near the surface of the reservoir to be discharged through this turbine.

When TVA started its Hydromodernization Program in 1990, it was determined that the first consideration for increasing DO at the various mainstem projects should

[1]Senior Environmental Engineer, Resource Group, Tennessee Valley Authority, 1101 Market Street (HB 2C), Chattanooga, TN 37402

[2]Environmental Scientist, Resource Group, Tennessee Valley Authority, 1101 Market Street (HB 2C), Chattanooga, TN 37402

be the effects of plant operations on the withdrawal zone characteristics. In 1991 and 1992, studies were conducted at six projects and significant potential for increasing DO level was found at each project. The studies were conducted for 2-3 days each year when the low DO normally occurs at each project. These studies included measuring DO and temperature (T) in the turbine discharges as well as obtaining vertical profiles of DO/T in the forebay of the different units. These data together with plant operational data and near-field geometry were then used to determine which turbine unit(s) discharged water having the highest DO levels and the reasons why these particular units had higher DO levels. This paper summarizes the results of these investigations and presents information on some of the factors that affect the degree of success one might expect to find with this approach at other projects.

<u>Factors that Affect DO In Mainstem Projects</u>

Two earlier papers (Ruane and Hauser, 1990 and 1991) presented information on the characteristics of mainstem reservoirs. These papers mainly presented information on the factors that affect the DO in the reservoir itself. Reservoir DO characteristics are greatly influenced by physical factors, and the most important one is the retention time of water within the project. Mainstem projects typically have retention times of about 25 days or less. Since mainstem projects have such low retention times, they are very sensitive to the variation in annual hydrology. During low flow years, the DO characteristics within the mainstem reservoirs can be dominated by in-reservoir processes such as sediment oxygen demand and the decomposition of organic matter. During wet years, the DO characteristics are often dominated by the DO concentrations in the inflow to the reservoir (Brown et al., 1985).

Another important characteristic of mainstem reservoirs is that the low retention times result in weak thermal stratification. The temperature regimes in mainstem projects are dominated by the inflow temperatures. Hence, stratification is usually strong only in the spring months, and as the flows continue through the spring and summer, the cooler water is released and replaced by the warmer inflow waters.

For the TVA mainstem reservoirs, the DO in the inflow to these impoundments usually dominates the DO in the reservoir during normal- and high-flow years, but during low-flow years, the in-reservoir processes dominate the DO (Brown et al., 1985 and Shiao et al., 1993). DO concentrations in the releases from these projects are usually higher during wet years and lower during dry years. The turbine discharges from the TVA mainstem projects reported in this paper all have median annual low DO values greater than 5 mg/L, but the minimum DO levels range from 1 to 3 mg/L. The characteristics of these mainstem projects are provided in table 1.

In considering ways to increase DO in the turbine discharges from mainstem hydro projects, it is important to consider the DO in the reservoir itself as well as the ways of improving DO in the releases. For example, upstream wastewater discharges can affect DO levels such as was found when the city of Knoxville improved its wastewater discharge quality to Fort Loudoun Reservoir and improved the DO in the releases from the reservoir by 1 mg/L. Also, the effects of DO in the inflow to the reservoir are important to consider. For example, when the DO in the releases from Boone Reservoir increased in the early 1970s, the DO in the releases from Ft. Patrick Henry (the next project below Boone) increased about the same amount. Finally, on an integrated reservoir system like TVA's, it is possible to consider maintaining a minimum weekly flow that is sufficient to maintain an improved DO level within the reservoir. An example of this approach is the maintenance of 13,000 cfs minimum

TABLE 1. CHARACTERISTICS OF THE SIX PROJECTS STUDIED

	KENTUCKY	PICKWICK	WILSON	WHEELER	GUNTERSVILLE	CHICKAMAUGA
Pool Elevation (ft)	359	414	508	556	595	682
Drainage Area (sq mi)	40,200	32,820	30,750	29,590	24,450	20,790
Volume (1000 ac-ft)	2,839	924	634	1,050	1,018	628
Surface Area (1000 acres)	160	43	16	67	68	35
Reservoir Length (miles)	184	53	16	74	76	59
Average Summer Flow (cfs)	40,300	34,600	32,800	33,000	28,300	27,300
Nominal Summer Retention Time (days)	35	13	9.7	16	18	12
Depth at Dam (ft)	88	82	108	66	65	83
Depth of Penstock Invert (ft)	77	67	38, 60	58	54	68
Depth of the Centerline Elev. of Wheels (ft)	59	55	31, 47*	46, 48	37	50
Height of Intake (ft)	45	36	15, 28.5	45	34	36
Generating Capacity (MW)	175	236	630	378	115	120
Number of Turbines	5	6	21	11	4	4
Total Turbine Discharge Capacity (cfs)	52,000	69,000	110,000	110,000	44,000	42,000
21-Year Median Annual Low DO (mg/L)	5.7	6.6	5.5	5.7	6.2	5.7
21-Year Minimum Low DO	2.3	2.4	1.1	2.6	3.2	2.4
Near-Field Features Affecting Withdrawal Zones	Submerged island upstream from units 1-3; bank next to unit 1; excavation for units 1 and 2	Bank next to unit 1; excavation for powerhouse adjacent to original river channel	Two intake levels; deep pool below intake levels	Spoil deposits adjacent to units 1 & 10; submerged islands of original river about one mile upstream	Bank adjacent to unit 1 and large shallow bay upstream from unit 1	Submerged island upstream from unit 4; large shallow bay upstream from unit 1

*The centerline of the intake elevation is reported for Wilson

weekly flow through Chickamauga to attempt to maintain an average DO of 4 mg/L (Hauser et al., 1990).

If it is determined that additional DO improvements are needed at a mainstem project, it is important to determine the near-field factors that affect DO at the project. The cost of aerating the turbine discharges for mainstem projects can be relatively high because of the large volumes of water that need to be aerated. The capacity for such an aeration system is dependent primarily on the amount of flow as well as the deficit in DO between the objective set for the releases and that which has occurred in the past or is anticipated to occur in the future. However, TVA's experience with mainstem projects suggests that designing aeration capacity for minimum DO is extremely conservative because minimum DO levels rarely occur. In 1989 TVA estimated the cost of aerating the releases from its mainstem projects and found that extremely high capital investments were required to meet the capacity needs for adding DO but that the annual operation costs were very low because the systems would be rarely used (Mobley et al., 1989). Because the annual operation costs were so much lower than the capital costs, it was decided to continue studies on the mainstem projects to see if operational changes could reduce capital costs or avoid adding aeration systems completely.

A major consideration in the studies was that although mainstem projects have weak stratification with respect to temperature (density), the stratification of DO in these impoundments was often great (e.g., 5-10 mg/L) when low DO concentrations occurred in the turbine discharges. It was thought that because of the weak stratification, changes in turbine operation might alter the stratification enough to make significant DO improvement. The weak thermal stratification which results in very little difference in the density of water offered the potential for flow through the mainstem projects to be manipulated by operations and to take advantage of near-field geometric features that might change the withdrawal zones from within the reservoir. For example, relatively small changes in reservoir bottom features such as submerged islands and the river bank as discovered at the Watts Bar project could be very significant in influencing the DO in the releases. Plant operations including which units and how many units are operated as well as the duration of operation of the units were thought to also be important factors that could change the withdrawal zone from within the reservoir.

Results of Field Studies at Six Hydro Projects

The objectives of the field studies were to determine the variability of DO at the project, to determine which units discharged water having the highest DO levels (under low and high flow conditions), and to determine the effects of multiple unit operations on increasing the DO. Also, the objectives were to determine the causes, if possible, for the DO variability found in the discharges from the project.

Table 2 summarizes the results of the studies. The hydrology for 1991 and 1992 was near normal, and on certain occasions the weekly flows during the studies were above normal. Originally it was planned to conduct the studies under the following conditions at each project: no turbine discharges for about ten hours each night before the studies began, begin operations with only one unit for three or four hours in the morning, and then bring on the other units to meet power demands for the remainder of the day. These conditions were rarely possible, so these studies were conducted typically under conditions as follows: 1 or 2 unit operation overnight, 1 or 2 unit operation in the morning hours for three to four hours, and then full project

TABLE 2. SUMMARY OF DISSOLVED OXYGEN MEASUREMENTS ON TURBINE DISCHARGES FROM SIX TVA MAINSTEM HYDRO PROJECTS (mg/L)

PROJECT AND DATES	RANGE OF MEASUREMENTS	MAXIMUM DIFFERENCES BETWEEN INDIVIDUAL DISCHARGES		MAXIMUM CHANGE IN DO FOLLOWING AN INCREASE IN NUMBER OF TURBINES OPERATING	MAXIMUM CHANGE IN DO OBSERVED AS A RESULT OF CHANGE IN FOREBAY DO
		LOW FLOW OPERATIONS	HIGH FLOW OPERATIONS		
KENTUCKY					
June 17-18, 1991	5.4-6.8	No Data	0.8 (Unit 1 > 5)	No Data	No Data
June 22-23, 1992	5.2-7.6	Insufficient Data	1.4 (Unit 1 > 5)	1.0 (Unit 1) ≈0 (Unit 5)	0.5 (High Flow)
PICKWICK					
July 23-24, 1992	6.0-8.2	Insufficient Data	0.8-1.1 (Unit 1 > 6)	1.5 (Unit 1) 0.7 (Unit 6)	0.5 (High Flow)
WILSON					
July 2-3, 1991	5.2-7.1	1.9 (Unit 4 > 19)	0.4 (Units 4, 8, 16 > 19 & 21)	-1.5 (Unit 4)	No Data
August 1-2, 1992	4.4-6.6	1.1 (Unit 1 > 21)	1.3 (Unit 1 > 21)	-1.4 (Unit 21)	0.6 (Unit 1)
WHEELER					
July 15-17, 1991	4.9-7.1	0.3 (Unit 11 > 1)	No Data	-1.1 (Units 1-5)	1.6 (Low Flow)
July 7-8, 1992	5.9-7.8	No Data	0.8 (Unit 10 > 4)	-1.2 (Unit 1) 1.2 (Unit 9)	None
GUNTERSVILLE					
July 22-25, 1991	5.4-7.9	0.6 (Unit 1 > 3 & 4)	No Data	1.0 (Unit 1)	1.1 (Low Flow)
August 4-5, 1992	6.5-7.7	Insufficient Data	1.0 (Unit 1 > 4)	1.1 (Unit 1)	No Data
CHICKAMAUGA					
July 15-17, 1991	5.5-7.4	Insufficient Data	0.5 (Unit 1 > 4)	1.0 (Unit 1)	1.9 (High Flow)
July 27-29, 1992	2.7-7.2	0.6 (Unit 4 > 1 & 3)	0.5 (Unit 4 > 1)	1.2 (Units 1 and 4) 2.7 (Unit 1)	1.0 (High Flow)

operations for the remainder of the afternoon. These conditions resulted in very little information being collected to measure the differences in DO between turbine discharges under low flow conditions. However, the data shown in table 2 in combination with the other data collected during the studies (i.e., the reservoir profiles and the information on near-field geometry) were sufficient to select the most likely units to discharge water having highest DO during low flows; but, additional data under low flow conditions are needed to confirm these selections.

The following discussions for each project summarize the reasoning for the results in table 2 as well as provide the rationale for selecting the units that would provide highest DO under low flow operations.

Kentucky. The DO in the discharges from units 1, 2, and 3 at Kentucky (unit numbers at TVA projects always start with unit 1 being the closest to the control room which is always located near a shoreline; therefore, the highest numbered unit is closest to the middle of the reservoir) is expected to always be greater than the DO in the discharges from units 4 and 5. This conclusion is based on the near-field geometry as well as reservoir profiles obtained during the studies. A submerged island is located just upstream from all three units, and the top of the island is only about 30 feet below the reservoir pool level. In addition, the river channel bottom is elevated in front of units 1 and 2 (elevation 280) compared with the elevation in front of units 4 and 5 (elevation 260) with the channel bottom being sloped from elevation 280 to elevation 260 in front of unit 3. The withdrawal zone for water for unit 1 is also influenced by an adjacent wall and a sloped abutment upstream from the wall that causes the withdrawal zone to draw more water from the top of the water column.

Pickwick. The Pickwick turbine units are located on an excavated overbank area adjacent to the original river channel. The elevation of the bottom in the forebay is elevation 340, whereas, the bottom elevation of the original river channel is elevation 332. Based on DO and temperature measurements in the releases as well as in the forebay, the discharge from unit 1 is influenced by the adjacent wall and sloped abutment causing the withdrawal zone to draw water from higher in the water column and resulting in higher DO levels in the discharge from unit 1. It would be anticipated that under low flow conditions that unit 1 would result in a higher DO level than the other units.

Wilson. Wilson is a relatively deep project and has no near-field geometric features above the project that influence the withdrawal zone. However, the releases from Wilson are affected significantly by the different depths of the intakes, with turbine units 1-18 having a depth of penstock invert of 38 feet and units 19-21 having a penstock invert depth of 60 feet. Hence, the turbine discharges from units 1-18 typically have considerably higher DO levels than units 19-21. It is interesting to note, however, that in 1991 as shown in table 2 the DO in unit 4 decreased by 1.5 mg/L following an increase in the number of turbines operating. This was due to the withdrawal zone expanding to a deeper depth within the reservoir and drawing water from where DO is lower. A similar observation was made on unit 21 in 1992. This condition could be alleviated by a four-month (June-September) operational strategy where the operation of units 19-21 is given preference when possible to reduce the retention time of water at the lower depths so that water at this depth will maintain higher DO levels. This operational scheme would not have to interfere with power generation to meet peak demands or high priority special operations for these units.

Wheeler. The Wheeler powerhouse is located on top of the original river channel and does not appear to have any obvious dominant near-field geometric features that directly affect DO in the turbine discharges. However, there are two spoil dumps located upstream from the units that could have an influence on the withdrawal zone, and even more remotely, there are a large number of submerged islands about one mile upstream from the powerhouse. These latter features probably do affect the DO in the turbine discharges under high flow conditions. Perhaps the most unique feature of the Wheeler project that influences the withdrawal zone is the high flow capacity (110,000 cfs) combined with the relatively shallow forebay. These two characteristics result in the withdrawal zone expanding to the surface of the reservoir to such an extent that an obvious surface water velocity can be observed within 50 to 100 feet upstream from the powerhouse. Based on the data collected in 1991 and 1992 it appears that the outer units, i.e., units 9-11, discharge water having the highest DO levels during high flow periods. It is interesting to note that units 1-5 during both years 1991 and 1992 experienced decreasing DO levels as the number of units operating were increased. This finding was anti-intuitive with respect to typical withdrawal zone phenomenon so other factors were examined for causing this DO decrease. One possibility for this is the occurrence of a significant amount of low DO water in a submerged pool upstream from the old shoals area about one mile upstream. As the discharge through the plant is increased, units 1-5 tend to draw water through the old river channel, whereas, the outer units tend to draw water more from the surface of the lake over the submerged shoals area thereby resulting in higher DO. On the other hand, it is conceivable that the discharges from an upstream paper mill could be influencing these results as well. Based on the data collected to date, it appears that the outer units result in higher DO levels, and based on the data in 1991, the outer units also would be expected to result in a higher DO level, although perhaps only marginally higher than the inside units. It should be noted that TVA's current monitoring strategy calls for weekly monitoring of the unit nearest the stream bank that is operating when the sample is taken. As a result historical data at Wheeler are biased on the low side since these studies show that the highest DO levels occur in the outer units, which have rarely been sampled.

Guntersville. Unit 1 at Guntersville, the unit next to the powerhouse, was found consistently to have higher DO levels than the other units. This was found to be true under low flow operations as well as high flow operations. Also, DO in unit 1 was found to increase consistently after multiple units were brought on load. Two near-field geometric features appear to explain why the DO is higher in unit 1: an adjacent abutment which has resulted in shallower depths in front of unit 1 as well as an adjacent bay area upstream from unit 1 which is somewhat shallower than the main river channel and probably provides a large volume of relatively high DO water that affects the withdrawal zone.

Chickamauga. Two geometric features affect the DO in the discharges from Chickamauga. Unit 4, and possibly unit 3, is influenced by a submerged island that is approximately a quarter of a mile upstream and varies between 20 and 30 feet deep. The island runs parallel to the old river channel. Unit 1, and possibly unit 2, is influenced by a large bay area which is only about ten feet deep that is adjacent and upstream from unit 1. As unit 1 is operated, some water from the bay is pulled into the old river channel and contains a higher DO level and thereby influences the DO in the forebay of unit 1 and possibly unit 2. As indicated in table 2 the results of Chickamauga appear to be conflicting, particularly under high flow operations, where at one time unit 1 had a greater DO than unit 4, and another time unit 4 had a greater DO than unit 1. However, based on analysis of the DO profiles collected from within

the reservoir in conjunction with the data collected in the releases, it is concluded that units 4 and 3 should be operated during low DO periods and that they will provide a more consistent higher DO level than units 1 and 2. Units 1 and 2 will occasionally have higher DO levels, but this will be when all units are running and DO will be relatively high anyway (about 1 mg/L higher than when only one or two units are running). Units 3 and 4 will stay higher in DO for a longer duration, whereas, the DO in units 1 and 2 will probably drop after only a few hours. Another observation worth noting was that on the morning of July 27, 1992, a DO concentration of 2.7 was measured in the releases from unit 1, while it was operating at about half load, and it was the only unit operating. This operation resulted in the least expansion of the withdrawal zone and allowed for water being drawn off the bottom of the reservoir without being diluted by water higher in the water column with higher DO levels. This occurrence was short lived, however, because generation was increased within the hour, and it is suspected that DO concentrations increased as well. Such operations are unavoidable but are short-lived, and impacts (if any) are probably limited to an area in the tailrace.

It is interesting to note the influence of the forebay DO on the DO in the releases as reported in the last column of table 2. Usually the change in the DO in the forebay profile was related to change in the oxygen level in the epilimnion (i.e., the surface water) and was caused by algal oxygen production. This factor alone contributes significantly to the DO variability seen in the releases from mainstem projects. This is an important consideration in developing DO enhancement measures.

Conclusions

These studies showed that the withdrawal zone at mainstem projects can be significantly influenced by near-field geometric features and operations of the hydropower project. Consideration of near-field features such as those found at TVA projects in conjunction with 2-3 day studies on DO and temperature profiles as well as collecting data on the turbine discharges can provide information for considering relatively low-cost operational changes to increase DO in the turbine discharges. Although more data on the TVA projects are needed under low flow operations, the field data that were collected under normal flow operations in conjunction with the near-field geometry provided sufficient information to select the most probable operational scheme to increase DO in the turbine discharges.

DO in the turbine discharges can be enhanced by using the following operational changes: first, by selecting the units that discharge water having the highest DO during low flow operations (i.e., during operations involving one or two turbine units); second, by operating when possible at the highest generation rates at each project (i.e., during low generation periods such as early summer mornings, obtain the desired generation from one project rather than from two or more projects). Such operational changes, where possible, will result in some amount of DO improvement at each project, some more than others, during the 1-3 month durations each year when low DO levels normally occur. In addition to increasing the magnitude of the DO levels, such operational changes will also decrease the frequency of low DO levels occurring. Operational considerations such as these can either eliminate DO concerns at selected projects or significantly reduce the cost of aeration schemes that may be needed to meet DO targets. The cost of aeration can be reduced because (1) the magnitude of DO improvement will be less, (2) the frequency of use will be less, and (3) only a few of the units probably will need aeration as opposed to all the units for the hydro project.

The results of these studies suggest that two aeration methods may have more promise for mainstem projects than other methods. One method that needs serious consideration is upstream near-field geometric modifications such as submerged weirs so that some of the influences that were observed during these studies can be replicated. Such methods would have the advantage of a one-time cost and very little operational cost. The other method that would appear to have promise would be the use of oxygen if capital cost can be held low. This method comes to mind because of the high variability shown in the need for DO and that in many situations and in some years DO enhancement would not even be needed. The variability in DO from hour-to-hour and day-to-day also suggests that oxygen may be promising because it would only be used as needed. Perhaps the combination of some near-field geometric modifications in conjunction with oxygenation could be the least-cost alternative for mainstem projects that normally do not have low DO levels below target objectives.

Appendix

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DISSOLVED OXYGEN ANALYSIS FOR HYDROPOWER ADDITIONS ON THE ILLINOIS RIVER

Mark J. Sundquist, P.E.[1]
Steven A. Elver, P.E.[2]

Abstract

The Illinois Waterway is comprised of a system of eight locks along the Illinois River, the Des Plaines River, and the Chicago Sanitary Ship Canal which allow commercial barge traffic between the Mississippi River and Lake Michigan at the City of Chicago. Opportunities for production of hydroelectric power is present at several of these lock and dams.

This paper presents the field study and computer simulation conducted to determine the feasibility of constructing hydroelectric powerhouses on two of these lock and dams, namely, the Brandon Road Lock and Dam and the Dresden Island Lock and Dam. Historically, the Illinois Waterway, particularly near metropolitan Chicago, has exhibited poor water quality. However, tremendous improvements in the past few years have occurred along the waterway. So as not to degrade these improvements, the Federal Energy Regulatory Commission (FERC), in issuing the construction and operating licenses for these two hydroelectric facilities, requires the hydropower additions to not reduce the dissolved oxygen (D.O.) level downstream of the hydroelectric facilities below 6 parts per million (ppm).

Presently, the waterway discharge passes through taintor gates at both of these lock and dam facilities which creates aeration. The addition of hydroelectric powerhouses would divert water from these spillways through generation equipment; consequently, the spillway aeration would not occur.

The purpose of the study was to determine the amount of power generation from these facilities, given the existing waterway water quality and the FERC D.O. criteria.

[1]President, STS HydroPower, Ltd., 111 Pfingsten Road, Northbrook, Illinois 60062
[2]Associate, STS HydroPower, Ltd.

A computer simulation generation analysis was conducted to provide a database of the waterway water quality. A four-month extensive field collection survey was conducted over the 63 kilometer (39 mile) reach of the waterway which comprises the two downstream pools of the Brandon Road and Dresden Island projects, and 3 kilometers (2 miles) upstream of the Brandon Road Project.

The analysis revealed that the hydroelectric additions were economically feasible and are an example of how the benefits of hydroelectric development can be balanced with environmental concerns.

Description of Illinois Waterway and Lock and Dam Facilities

A map of the waterway is shown in Figure 1. Near the Chicago area, the waterway consists of a combination of ship canals and local rivers such as the Des Plaines River and Chicago River. These connect south of Joliet to form the Illinois River, which then comprises the remainder of the waterway to the Mississippi River. The entire length of the waterway is approximately 563 kilometers (350 miles).

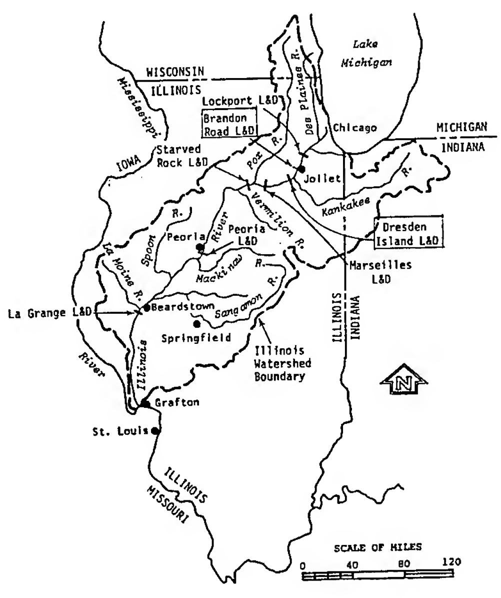


Figure 1. Illinois Waterway

The Village of Rockdale, Illinois received a FERC license for the construction and operation of a hydroelectric facility on the Brandon Road Lock and Dam located approximately 65 kilometers (40 miles) southwest of Chicago. The lock and dam is operated by the U.S. Army Corps of Engineers and is located on River Mile 286 of the Illinois Waterway system.

The Brandon Road Dam consists of 21 taintor gates of 0.70 m (2.3-foot) height, two uncontrolled ogee spillways, and 16 head gates - eight of which were decommissioned in 1987 by filling with concrete. The project proposes to install four horizontal rim-drive turbines of 1500 KW each, for a total project capacity of 6 MW. The facility will have a hydraulic head of 9.1 m (30 feet).

The Dresden Island Lock and Dam is located 13.5 river miles downstream of the Brandon Road facility, at River Mile 271.5. The Village of Channahon, Illinois received a FERC license to construct and operate the hydroelectric addition to the lock and dam facility. The dam consists of 9 taintor gates of 6.4 m (21-foot) height, 2 uncontrolled ogee spillways, and 18 head gates which were decommissioned in 1982 by filling with concrete. The facility is also operated by the U.S. Army Corps of Engineers. The FERC license allows the construction of a 10 MW generating facility. The facility is planned to be installed with 8 horizontal rim-drive turbines. The facility has an available hydraulic head of 5.2 m (17 feet).

The next lock and dam structure downstream from the Dresden Island facility is the Marseilles Lock and Dam at River Mile 247.0. The lengths of the downstream pools are 23 kilometers (14.5 miles) for the Brandon Road facility and 39 kilometers (24 miles) for the Dresden Island facility.

Description of Analysis

The determination of power generation from a hypothetical hydroelectric powerplant is easily analyzed if a historic discharge record is available. Long-term flow records are available on the Illinois Waterway from both the U.S.G.S. and the Chicago Metropolitan Sanitary District. If water quality is not an issue, the expected generation from the powerplants based on the historic discharge database can be calculated by the straightforward analysis of the efficiency and discharge characteristics of the expected turbine/generator and by incorporating the project's headwater and tailwater curves. However, including water quality consideration provides an added complication to the analysis.

In accordance with the FERC licenses, the Brandon Road and Dresden Island projects can not be operated in such a manner that the D.O. level in the downstream pools are less than 6 ppm. To integrate the water quality aspects into the power

generation analysis, the seasonal D.O. levels in the waterway, in addition to the aeration efficiencies of the existing spillways at the dam spillways, must be known. An extensive field study to determine the existing D.O. levels in the waterway was conducted during the Summer and Fall of 1991.

The water quality criteria was included into the generation analysis computer model as follows. From the field study, monthly average D.O. levels for the upstream reservoir pools were included in the analysis. The power generation study utilizes daily average flows in the waterway. For each day, the downstream D.O. was computed as being the straight-forward mix, in relationship to the discharges, of flow going through the powerhouse and flow going through the dam spillway. The D.O. level of the powerhouse spillway was assumed to be equal to that of the upstream reservoir, as no aeration of the turbine discharge occurs. Aeration of the flow occurs as it passes through the dam spillways. The efficiency of the spillways as aerators were studied as part of the FERC license applications. Both the Dresden Island and Brandon Road spillways acted as excellent aerators with near 100% saturations occurring below the dam spillway. Combining the spillway aeration efficiency, in combination with the average water temperature for the month would result in the D.O. levels immediately below the dam spillway.

Both the Brandon Road and Dresden Island facilities use multiple turbine/generator arrangements. In the analysis, as a first step, it was assumed that the powerhouse would draw as much discharge as available under run-of-river operation to run the turbine/generators. If the D.O. level downstream of the powerhouse was below 6 ppm, a single turbine or multiple turbines were assumed to be shut off and that corresponding discharge passed through the spillway until the proper mix occurred such that the downstream D.O. level would be above 6 ppm.

Field Study

An extensive field study was conducted during the Summer and Fall of 1991 to physically measure the D.O. and temperature of the waterway. The D.O. readings were taken starting upstream of Brandon Road Lock and Dam at River Mile 288 and terminated at the end of the Dresden Island downstream pool at River Mile 247. This encompasses a 66 km (41-mile) reach of the waterway. Along this stretch, 38 locations were identified for the taking of D.O. readings. D.O. readings were taken at approximately two-week intervals beginning on June 4, 1991 and completed on September 30, 1991.

The readings were taken with a portable D.O. meter. The meter used was a YSI Model 57, frequently calibrated for accuracy. The readings were taken travelling the waterway by outboard motorboat. Typically, readings were begun

around 8:00 a.m. and completed around 5:30 p.m. (At two locations, on two occasions, hourly readings were taken over a 24-hour period to determine the extent of D.O. fluctuation on a daily cycle. Both sets of readings show no change between day and night readings.)

Attached Figures 2 and 3 are examples of the data collected along the waterway for the month of June and July, 1991, respectively. The D.O. levels upstream of Brandon Road are approximately 4 ppm. D.O. levels downstream of the Brandon Road project, to the Marseilles Lock and Dam remain at near saturation levels. The D.O. field study allowed a table of monthly averages, D.O. levels to be input to the computer program.

Results of Analysis

Daily waterway discharge records are available for the period of 1940 to 1990 for the Brandon Road Project and from 1944 to 1989 for the Dresden Island Project. The analysis resulted in the following information.

Based on the historic flow records, the Brandon Road Project produced an average annual generation of 40.4 GWH when considering the water quality criteria. If there had been no water quality criteria, the project would produce 48.5 GWH. Consequently, the D.O. criteria results in a 17% reduction in potential generation. Power generation was restricted to some extent on the average of 93 days per year.

For the Dresden Island Lock and Dam, the project would produce 61.3 GWH considering the water quality criteria. Without the water quality criteria, the total potential annual generation of the project is 68.7 GWH. Consequently, the D.O. criteria results in a reduction of 11% of generation from the available potential. Power generation was restricted to some extent on the average of 71 days per year.

	Brandon Road	**Dresden Island**
Power generation with W.Q. criteria	40.4 GWH	61.3 GWH
Power generation without W.Q. criteria	48.5 GWH	68.7 GWH
Number of days per year generation is curtailed	93 days	71 days

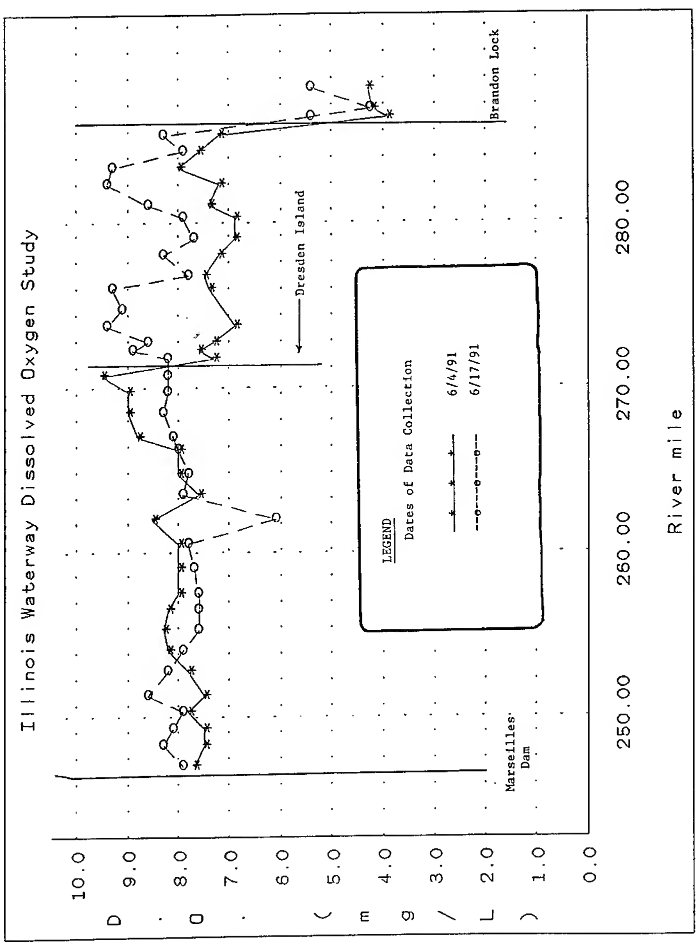


Figure 2. Dissolved Oxygen Readings for June, 1991

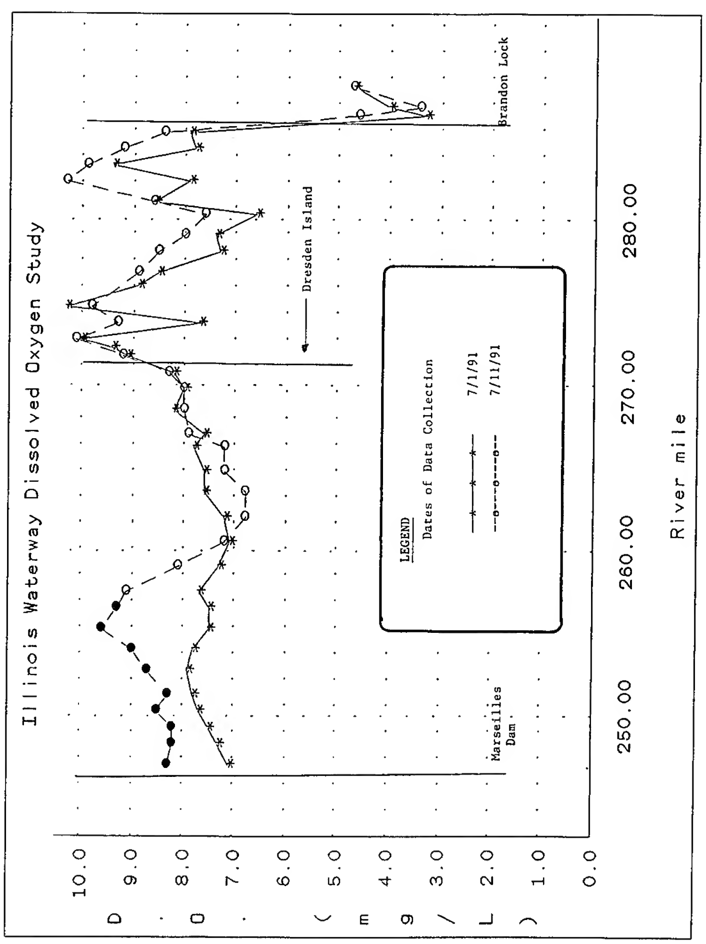


Figure 3. D.O. Oxygen Readings for July, 1992

Although the water quality criteria does result in reduction of the electrical generation, the projects are still economically feasible at typical industry standard avoided costs from the utilities. The hydroelectric projects reveal that the addition of these hydroelectric powerplants to the Illinois Waterway will not result in any degradation of D.O. levels. The resulting benefits of pollution-free, renewable energy, in addition to the investment into Illinois and the associated fishing and park recreational areas which are required to be constructed with these facilities, are benefits which the citizens of Illinois will be able to enjoy for years ahead.

HYDRO TURBINE AERATION

Brian S. Greenplate*
Joseph M. Cybularz**

Abstract

Traditionally, hydro equipment manufacturers have supplied the industry with turbines designed to optimize performance and cavitation resistance for site specific conditions. Awareness of environmental concerns and water quality has prompted manufacturers and power producers to begin considering methodology which will allow the turbine to admit air to the flow. Admission of air increases the dissolved oxygen level of the flow, which translates to improved water quality. The concept of turbine air admission presents both technical and commercial challenges to the manufacturer. The technical challenges deal with the prediction of 1) performance effects due to air admission, 2) dissolved oxygen uptake and 3) maximum permissible air flow through a turbine. The commercial challenges are centered around how to best implement air admission through the turbine, while minimizing cost and outage time.

* Program Manager - TVA Hydro Modernization, Voith Hydro, Inc., P.O. Box 712, York, Pennsylvania 17405, Telephone: (717) 792-7000

** Manager, Hydraulic Technology, Voith Hydro, Inc., P.O. Box 712, York, Pennsylvania 17405, Telephone: (717) 792-7000

I. Introduction

In recent years, federal legislation[1] has been enacted in an attempt to regulate dissolved oxygen (DO) concentrations at hydroelectric facilities. The legislation may require that a hydroelectric powerhouse have the ability to improve water quality by increasing the DO levels discharging from the turbine. Low DO levels occur in many reservoirs for a variety of reasons. Taking advantage of the low pressure regions of a hydraulic turbine to naturally aspirate atmospheric air is a solution to the problem.

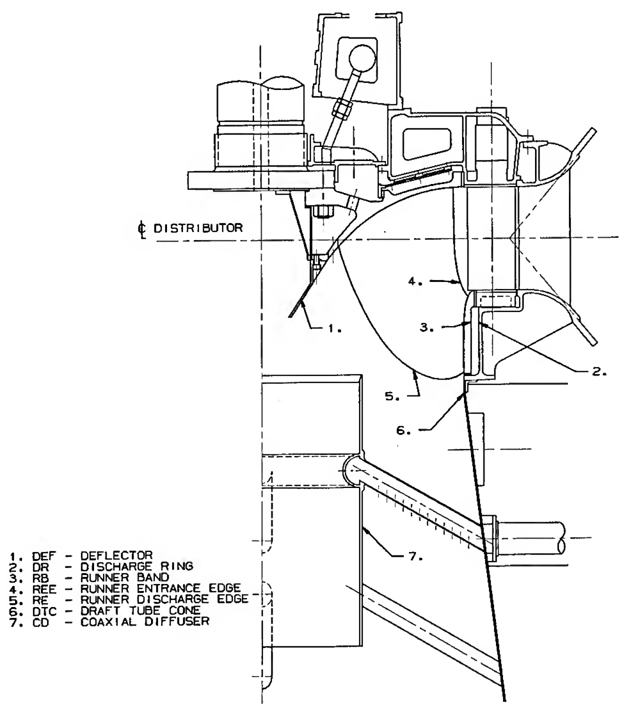


Figure 1
Possible Low Pressure Air Admission Locations on a Francis Turbine

In the vast majority of hydroelectric facilities, a sub-atmospheric pressure exists in the region of the runner discharge edge. As a result, air can be drawn into the turbines, such that mixing will occur with the flow in these low pressure regions. As shown on Figure 1, the location of the air

admission can be at several positions in the low pressure region of the turbine. As the air mixes with the water, a portion of the atmospheric oxygen will dissolve into the water, thus increasing DO concentrations discharging from the powerhouse.

Development of this type of turbine dates to the late 1950's per March et al. [2] and has been recently described as an auto-venting turbine (AVT). Both fixed and variable costs are associated with the implementation of an AVT for a particular site. The fixed costs are the one-time costs associated with purchasing the necessary equipment required for air admission. The variable costs are related to the lost revenue that may be incurred due to performance degradation of the turbine and the maintenance required for the aerating components.

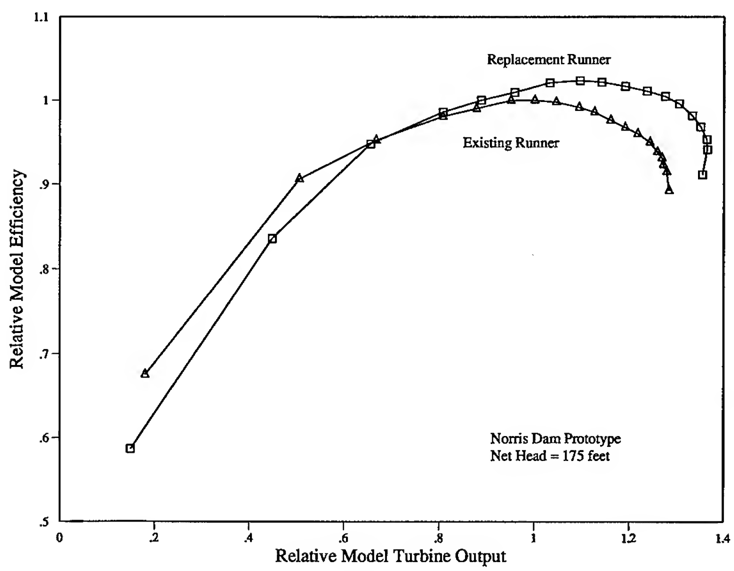


Figure 2
Comparison of Norris Dam Model Performance

While the fixed costs described above are relatively easy to quantify, the variable costs are extremely difficult to estimate at this stage of AVT evolution. Likewise, it is also difficult to predict the DO uptake potential for a given project. This paper will analyze the uncertainties of determining the variable costs and benefits from a technical point of view, and address frequently asked commercial questions regarding AVT.

II. Technical Issues

Technical issues were addressed on a joint development project between the Tennessee Valley Authority (TVA) and Voith Hydro. The project included the homologous model testing of the existing runner and an upgraded replacement runner for TVA's Norris Dam. The units at Norris Dam are medium specific speed Francis turbines rated at 66,000 horsepower under at a net head of 165 feet operating at a speed of 112.5 rpm. Figure 2 presents a performance comparison between the model results of the existing and replacement runner at the corresponding prototype net head of 175 feet.

AVT presents several new technical challenges to the turbine manufacturer. These challenges are defined as follows:

1. Performance effects due to air admission.

2. The prediction of the increase in DO uptake.

3. The prediction of the quantity of air that would be naturally aspirated through the turbine.

The performance effects are needed to determine the variable costs associated with the benefits of DO uptake. The increased DO uptake and quantity of air naturally aspirated through the turbine quantifies the benefits of AVT.

Performance Effects

The major mechanisms which affect performance during air admission on a typical hydroelectric facility have been estimated by Almquist et al. [3]. The mechanisms that would dissipate energy are related to hydraulic, buoyancy, drag, compression and shear losses. The hydraulic losses are due to the increase in velocity at a given discharge due to the volumetric addition of air. To overcome the hydrostatic pressure gradient (buoyancy) the air experiences in the vertically downward sections of the water passageways, energy is expended, thus creating losses. A drag loss will occur due to the relative velocity difference between the water and air bubbles. As the air travels from the low pressure region around the turbine to the higher pressures at the discharge of the powerhouse, energy must be transferred to compress the air bubbles. Further, as the air bubbles are sheared, smaller bubbles are created thus expending additional energy.

At this time, a numerical calculation of losses using a fully developed three-dimensional solution incorporating the appropriate first principle governing equations is not available. Using simplifying assumptions, one can calculate some of the losses. As an example, the technique used by Almquist et al. predicts the losses, at one operating condition and one location for air admission, to be within 0.13 percentage points of the tested turbine efficiency differential with and without air admission. However, the Norris Dam model test results indicate that performance effects are also a function of the air admission location. Shown on Figure 3 are the Norris Dam model results for various aeration locations at the corresponding prototype net head of 175 feet and maximum wicket gate opening.

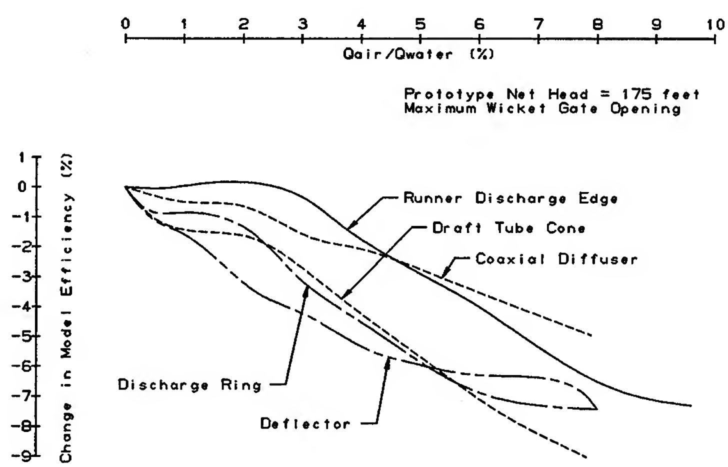


Figure 3
Comparison of Performance Effects from Norris Dam Model Tests for Various Air Admission Locations

The only avenue available to "economically" determine, at least on a relative basis, the performance effects at various air admission locations and turbine operating points is through model testing or extrapolation of tested data. The current industry standard model test codes do not address the methodology of testing air/water mixtures. The unique problems of model testing have been outlined by Mobley and Brice [4]. It is suggested that Froude number similarity should be used to determine a model test speed. Further, the model net positive suction head should be such that the model and prototype static pressure are equal at the mid-point of the draft tube vertical section.

DO Uptake

Prediction of DO uptake is the most difficult of the three technical issues previously mentioned. DO uptake depends on many factors. The primary factors are the upstream DO level, the ambient DO saturation level, the pressure field the mixture experiences, the time lapse between flow passing the air admission location and the downstream DO monitoring point, bubble size, and turbulence. A numerical analysis tool for DO uptake has been developed [5]. However, this numerical model does not address several of the above parameters and thus is still under development. With this in mind, a proposal has been submitted to the ASCE Hydraulics Division to form a task committee which will study and report on this problem.

Until this report is issued, model testing DO uptake would appear to be the only reliable solution to obtaining data. Results of such testing are reported by Brice and Cybularz [6] on the Norris Dam model test.

Air Flowrates

The quantity of air that can be aspirated into the turbine discharge from the atmosphere is a function of the pressure at the air admission location and the dynamic losses that exist in the air delivery system. Predictions of the pressure at the air admission location, without air admission effects, can be made by existing numerical codes. Model testing can provide more accurate data for most air locations and air flowrates. For air admission points located on the rotating components such as the deflector and runner, very special instrumentation is required to minimize the rotational pressure effects. In addition, pressures can be measured on the prototype equipment using pressure transducers.

The dynamic losses of the air delivery system can be calculated using simple one-dimensional loss analysis on some of the simpler air manifolds. However, these calculations are not normally within a turbine

manufacturer's expertise. For example, the air passageways required to deliver air to the runner discharge edge are very complex. Accurate predictions of maximum air flow requires a detailed analytical air model.

III. Commercial Issues

When considering the possibility of utilizing an AVT to increase the DO levels of flow exiting the turbine, there are typically several commercially related questions raised. The following items address some of the most prominent questions:

1. Which method or combination of methods are best suited for a particular site?

   This is 1) a function of the type of turbine which exists at the plant being considered for air admission, and 2) the pressures existing at the potential air induction locations.

   Francis and fixed blade type turbines are candidates for all of the air admission locations described in Figure 1. Kaplan turbines, on the other hand, are not practical for considering discharge edge and deflector air admission due to the mechanical mechanism required within the hub.

   A final consideration in determining the methods of air admission, is the required levels of DO uptake for the turbine. Depending on the pressures existing at the admission locations, it may require a combination of several induction locations to achieve the desired level of DO.

2. Should AVT technology be incorporated as part of an upgrade or rehabilitation?

   This issue is a function of the types of methods that will be employed to increase the level of DO exiting the turbine. If discharge air admission is used, a new runner, containing air-passages, would be required in order to transport air to the discharge edge of the buckets. In this scenario, it is economically logical to purchase an upgraded modern design runner, complete with discharge edge air admission capability. In most cases, the increased turbine performance will significantly aid in offsetting the cost of the aerating runner. The other methods of air admission do not require runner replacement, but would need an outage to install the aerating equipment. The outage length for installation of the discharge

ring and draft tube cone methods is quite significant, as it requires complete disassembly of the unit and significant concrete excavation. Depending on the condition of the machine, it may be advantageous to perform rehabilitation of the unit during the outage. Of the methods available for air-admission, the deflector represents the simplest implementation, provided that adequate air-passages exist in the turbine for transmitting air to the deflector.

Obviously, if aeration is considered for a new installation, outage and downtime is not a concern and turbine aeration can be fairly easily implemented.

3. Is model testing required, and if so, what is the schedule and cost impacts?

As discussed previously, in order to understand which methods of aeration are best suited for a given plant, it is important to understand the operating pressures in the turbine. This data can be obtained by either model testing or prototype field testing. If model testing is selected, it is logical that it would be performed in conjunction with performance testing. The price increase for performing a model test involving aeration testing (i.e.performance effects, DO uptake measurements, pressure readings, etc.) would be approximately 40% above the cost of a normal model test. The model test schedule would increase by approximately six weeks beyond the normal model test duration when including aeration testing in the scope.

4. What are the cost and schedule impacts on the supply of prototype AVT equipment?

When considering AVT equipment, cost and schedule are obviously important factors. Costs are a function of the methods utilized to admit air to the turbine. Voith Hydro is currently in the process of manufacturing its first AVT components for the Norris Dam project. Consequently, there is very little cost experience from which to draw. However, it is possible to provide rough information on the four most prominent methods of admitting air to the turbine. The approximate price increase for procuring a runner with the capability of admitting air through the discharge edge of the buckets and through the deflector is approximately 40%

above the price of a normal new Francis runner. The price for procuring either the discharge ring or draft tube cone aerating equipment (uninstalled) is approximately 15% of the price of a normal new Francis runner. Based upon Voith Hydro's current experience with Norris Dam, it is estimated that the delivery schedule for a runner capable of aerating through the buckets and deflector would be approximately two to three months longer than for a normal Francis runner.

5. How will performance and DO uptake guarantees be addressed for AVT?

As previously discussed, performance effects from inducing air into the flow and DO uptake have been measured in the laboratory. However, the corresponding prototype measurements will not be made at Norris Dam until the summer of 1995. Until these measurements are taken, it is difficult to project the correlation between the model and prototype. Even once this data is gathered, there will still be many unknowns regarding AVT. As a result, significant analysis and research is still required before it is possible to accurately predict prototype performance during aeration or prototype DO uptake levels.

IV. Summary

While it is obvious that AVT technology is still in its early stages of development, significant advances are being made. The results of Norris Dam model testing have provided significant insight on turbine air admission. It is anticipated that the prototype DO related results from Norris Dam will lend even further advances. Without a doubt, the industry, at urging and guidance of power producers such as TVA, will continue to make strides in turbine air admission.

[1] Fuss, J.T. and D. Rastler, "Utility Options for Meeting Dissolved Oxygen Limits for Hydroelectric Power Plant Discharges", EPRI Draft Report Research Project 2694-8, Palo Alto, CA. February 1990.

[2] March, P.A., J.M. Cybularz and B.G. Ragsdale, "Model Tests for the Evaluation of Auto-Venting Hydroturbines," ASCE National Conference on Hydraulic Engineering and International Forum on Ground Water, Nashville, TN, August 1991.

[3] Almquist C.W., P.N. Hopping, and P.A. March, "Energy Losses Due to Air Admission in Hydroturbines," ASCE National Conference on Hydraulic Engineering and International Forum on Ground Water, Nashville, TN, August 1991.

[4] Brice, T. and J.M. Cybularz, "Air Admission Effects on Hydraulic Turbines," ASME Winter Annual Meeting, Anaheim, CA, November 1991.

[5] Mobley, M. and T. Brice, "Experimental Difficulties Encountered in Testing Air/Water Mixtures," ASCE National Conference on Hydraulic Engineering and International Forum on Ground Water, Nashville, TN, August 1991.

[6] Hopping P.N., "Numerical Modeling of Air/Water Mixtures for Internal Flows," ASCE National Conference on Hydraulic Engineering and International Forum on Ground Water, Nashville, TN, August 1991.

A Process for Selecting Options to Improve Water Quality Below TVA Hydro Projects

By J. Stephens Adams[1] and W. Gary Brock[2]

Abstract

TVA is making improvements in water quality and aquatic habitat for 16 TVA project tailwaters as a part of its Lake Improvement Plan (LIP). Options are being implemented to increase minimum flow rates and aerate project releases to raise dissolved oxygen (DO) levels. A process has been developed to facilitate planning efforts to achieve broad-based support for the recommendation and implementation of the most appropriate option for each project within the aggressive LIP schedule constraints.

This paper presents details of the planning process and TVA's experience in utilizing this process for projects under the LIP. The planning process is a team effort involving staff representing various program interests. All issues relevant to evaluation of options are identified and discussed early in the planning effort, resulting in a more complete analysis and avoiding surprises and unnecessary delays later on. In work group meetings, all participants have the opportunity to make specific concerns known, and also become aware of all issues considered. Coordination with State and local interests is also done early in the planning process to achieve area input and support for

[1]Senior Engineer, TVA Engineering Laboratory, Post Office Drawer E, Norris, TN 37828

[2]Projects Manager, TVA, Water Resources, Knoxville, TN 37902

project recommendations. The result of the planning process is a recommendation of an option for implementation that has the support of all involved.

A work group begins the process by identifying numerous potential options, and then narrowing the options to a manageable number. More detailed studies are performed for each remaining option. Studies include an economic analysis consisting of estimates of capital costs, annual operation and maintenance costs, and impacts to the TVA power system. Equally important is an evaluation of other considerations which are not easy to put a dollar value on including potential impacts to the environment, archaeological and historical resources, aesthetics, safety, reliability, flexibility, and other issues. Based on the results of the economic analysis and consideration of other issues, a recommendation for project implementation is agreed upon.

Examples of successful utilization of this planning process to select options for implementation at TVA's Blue Ridge project are presented.

Introduction

In 1991, the Tennessee Valley Authority (TVA) began implementation of its Lake Improvement Plan (LIP). This program was created to provide higher lake levels for increased recreation benefits, and to improve the water quality of hydropower plant releases. Under the reservoir releases component, 16 projects were targeted for improvements to provide minimum dissolved oxygen (DO) levels. The majority of these also required provision of minimum flows. Plans are to implement options providing the desired improvements at all 16 projects within five years.

This paper addresses the planning process used to identify, evaluate, and select appropriate aeration and minimum flow options for implementation at LIP projects. A thorough planning process must be conducted for each project to account for site-specific features and particular project conditions, restrictions, and operating policies.

A brief overview of the planning process will be presented, followed by examples from the study conducted for TVA's Blue Ridge project.

Planning Process

The LIP is managed by a task force led by staff from TVA's Water Management office. The task force meets periodically and directs all work to be performed under the LIP.

Planning studies for LIP projects are conducted using a group approach. At the beginning of a specific study for most projects, a subgroup of the task force is formed. The subgroup is composed of TVA staff with expertise in all areas relevant to the specific project. The purpose of the subgroup is to identify and evaluate potential aeration and minimum flow options, and to select the most appropriate option for recommendation to the LIP task force for implementation.

Group Approach

Many advantages have been realized by utilizing the group approach. The diverse composition of the subgroup provides a look at the project from the perspective of all program participants. This helps in assuring that all viable options are identified, leaving a minimal chance that a good option has been overlooked. In addition, all relevant issues associated with the project can be identified and addressed early in the planning process, hopefully eliminating surprises and unnecessary delays later on. The group approach also results in widespread support for the final subgroup recommendations. This is attributed to the fact that all participants are part of the process from the beginning, have the opportunity to provide their input, and hear all the arguments associated with all options. Utilizing the group approach also helps to maintain good communication among TVA staff and organizations.

The subgroup also actively seeks to obtain input during the planning process from interested outside sources. Appropriate State staff are consulted to keep them abreast of LIP plans and activities, to obtain information on their objectives at specific projects, and to determine their preferences for implementation options. In addition, local officials are also contacted to obtain their input. Some State staff have attended LIP task force meetings when decisions on implementation options are being made.

The following is a listing of program areas having staff included in a typical subgroup.

- Project management
- Hydroplant management
- Power system operation
- Mathematical modeling
- Research and testing
- Water quality
- Fisheries
- Design and construction
- Cultural resources
- Recreation

Planning Process Steps

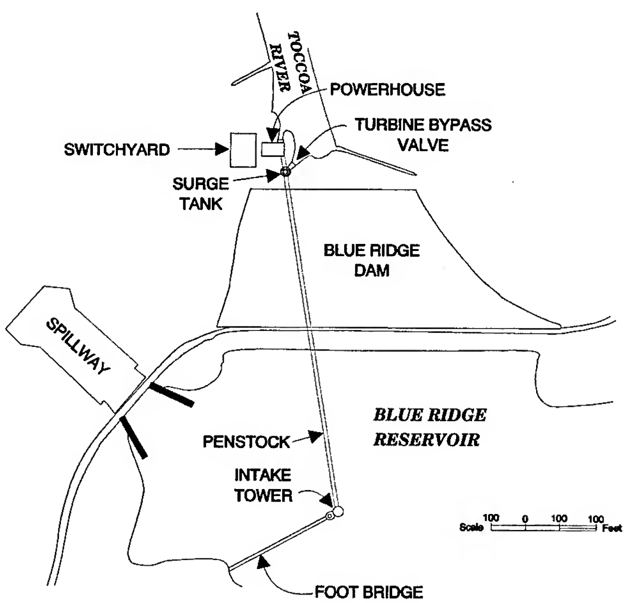
The following list presents the basic steps used for conducting the LIP planning studies. The remainder of the paper will describe each of these steps in more detail along with examples from recent planning studies for TVA's Blue Ridge project.

- Identify project objectives
- Identify potential options
- Narrow the number of options using a screening process
- Prepare cost estimates
- Evaluate intangibles
- Select and recommend most appropriate options for implementation

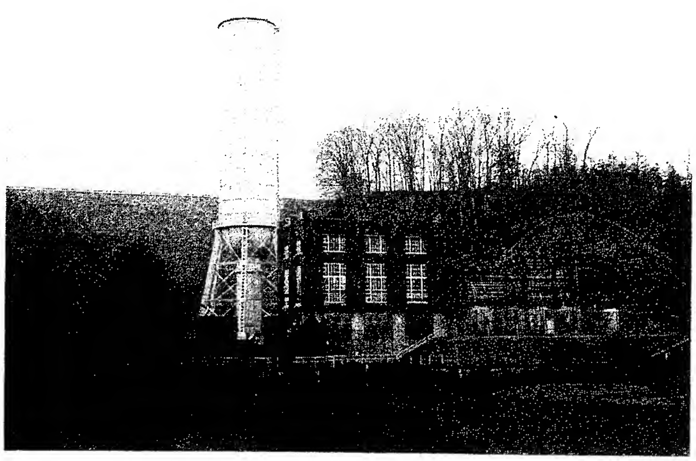
Background on Blue Ridge Project

TVA's Blue Ridge project is located on the Toccoa River in Fannin County, Georgia, at river mile 53.0. The dam was closed in 1930, and commercial operation of the power generating unit began in 1931. The dam is an earthfill structure with a maximum height of about 167 feet and a length across the top of about 1000 feet. An intake tower in the forebay connects by a 1050-ft long tunnel to a powerhouse located at the toe of the dam on the left bank. The powerhouse contains a single generating unit rated at 20 MW. The turbine is a vertical Francis unit directly connected to an open, air-cooled generator. Project features are shown in the following photograph and site plan.

Operation of the Blue Ridge project is typical of TVA's tributary projects. The reservoir is historically drawn down in the late summer and fall for flood control purposes and filled again in the late winter and spring. Actual daily releases for power generation vary depending on system load requirements and reservoir levels.


TOCCOA RIVER
POWERHOUSE
TURBINE BYPASS VALVE
SWITCHYARD
SURGE TANK
BLUE RIDGE DAM
SPILLWAY
BLUE RIDGE RESERVOIR
PENSTOCK
INTAKE TOWER
Scale 100 0 100 100 Feet
FOOT BRIDGE

SITE PLAN



Minimum releases are now provided by operating the generating unit approximately two hours per day. This operation can cause very low flow conditions for several hours per day in the upper tailwater section. Minimum DO concentrations in project releases occur between July and October, and can be as low as 3 mg/L.

Identify Project Objectives

TVA prepared an Environmental Impact Statement (EIS) for the LIP, and objectives regarding minimum flow and DO levels are specified. Options considered for each project must meet these objectives. Release objectives for the Blue Ridge project are a minimum flow of 115 cfs and minimum DO of 6 mg/L.

Identify Potential Options

One of the first tasks in the planning process is to identify all potential minimum flow and aeration options to be considered. This is done by compiling a listing of all possible options, regardless of perceived feasibility, and noting advantages and disadvantages. This provides a basis for a review of each option.

A full range of options are considered for providing minimum flows and improving DO. The following is a listing of typical options considered.

Minimum Flows Options:

- reregulating weir
- small generating unit
- turbine pulsing
- sluicing

Aeration Options:

- aerating weir
- oxygen injection
- compressed air
- turbine venting
- surface water pumps
- autoventing turbines

Narrow Number of Options

The list of potential options is usually much too long for a detailed evaluation of each option to be conducted. It is preferable to reduce the number of options to about four or five if possible using a screening process, and study these further.

In the first subgroup meeting, each option is briefly discussed to determine whether further investigations are warranted. Sometimes more than four or five options are selected during this meeting, and further study is required to narrow the options to the desired number.

An important objective in the first subgroup meeting is to discuss existing conditions at the project and in the tailwater downstream. These discussions are intended to provide all participants with a good understanding of project features, operating policy, purposes, water quality, tailwater uses and other considerations so that all relevant issues can be identified and addressed early in the process. This information is also used to avoid consideration of options which would adversely impact existing project purposes and uses.

For the Blue Ridge project, minimum flow options selected for more detailed studies were a small unit installation and a reregulating weir. If both of these options proved to be unsatisfactory, backup options of spilling through the existing turbine bypass valve or pulsing with the existing generating unit would be considered.

Aeration options selected for Blue Ridge included oxygen injection, compressed air or turbine venting, and an aerating weir. The subgroup determined that more studies were needed to select the most appropriate location for oxygen injection, more investigation including field tests were necessary to evaluate venting and compressed air options, and additional analysis was needed to determine the best design and location for the aerating weir. Further review narrowed the options to a labyrinth aerating weir which would achieve both the minimum flow and aeration objectives, and a small generating unit in combination with either oxygen injection in the forebay or compressed air for both the small and existing generating units.

Prepare Cost Estimates

Cost estimates are prepared for each of the remaining options. Costs estimated include capital costs, operation and maintenance expenses, and power system impacts.

The following table presents the costs for the Blue Ridge project options.

Blue Ridge Minimum Flow and Aeration Cost Estimates

	Labyrinth Weir	Small Unit with O_2 Injection	Small Unit with Forced Air
Construction costs:			
-total capital costs	$3,280,000	$2,986,000	$3,121,000
Annual costs:			
-int. & amort.	$ 418,000	$ 381,000	$ 398,000
-O&M costs	39,000	39,000	10,000
-power system costs	21,000	7,000	46,000
Total annual costs	$ 478,000	$ 427,000	$ 454,000

Total capital costs include expenses for design, procurement, and installation of each option including any land purchase requirements. Interest and amortization costs are the annualized value of total capital costs assuming a 25-year evaluation period and a discount rate of 12 percent. Operation and maintenance (O&M) costs are the annualized value of O&M expenses expected to be incurred over the evaluation period. Power system costs are the annualized value of TVA power system impacts expected to be incurred by operation of each option. These impacts include capacity losses, lost energy generation, and shift of energy from peak to offpeak periods.

Evaluate Intangibles

Even though costs are important, intangibles, or items difficult or impossible to put a dollar value on, are also important considerations in option selection. The

following is a typical list of intangibles considered in the selection evaluation.

- Safety
- Water quality
- Reliability
- Public interest
- Private land impacts
- Cultural resources impacts
- Miles of tailwater improved

Select Most Appropriate Option

The last subgroup meeting is convened when all information needed for selecting the most appropriate minimum flow and aeration options has been obtained. The goal of the selection process is to select the most appropriate minimum flow and aeration option for implementation. Even though cost is a very important issue, options for some projects have been selected which were not the least cost due to other important intangible considerations. Each option is thoroughly discussed and options are selected for recommendation to the LIP task force for implementation. At the next formal meeting of the LIP task force, the subgroup recommendation is presented and a final decision made.

Blue Ridge Selection

Option selections at Blue Ridge had been narrowed to a labyrinth aerating weir and a small generating unit in combination with either oxygen injection in the forebay or compressed air for both the small and existing generating units.

Compressed air was eliminated due to its high costs and aeration performance uncertainties. This site was not as good as other sites TVA has evaluated for compressed air, and any design would involve a fair amount of guesswork.

The labyrinth weir option was also eliminated. Even though its costs were comparable with the small unit option, land issues were strong disadvantages. The weir site is on private land, about one mile downstream from current TVA property boundaries. The purchase of over 40 acres would be required, and about 20 landowners would be affected. An advantage of weir options is that they attract the public to the site. However at Blue Ridge, this advantage would be lost since access would remain

private. Property owners were not receptive to public access on their lands.

The small unit in combination with oxygen injection in the forebay was selected as the most appropriate option to provide minimum flows and aeration of turbine releases. This option has a slight cost advantage over its competitors, and would be constructed entirely on TVA property thus avoiding any private land concerns associated with the weir option. The small unit would provide a free flowing river type environment for the entire length of the tailwater to Ocoee No. 3 Reservoir. The only drawback is the potential adverse impact on historical structures. However, it is felt that more detailed design studies will minimize any potential impacts.

Conclusion

The planning process described in this paper has been successfully utilized by TVA for several of its LIP projects. Probably the most important factor contributing to this success is use of the group concept in the entire planning process. Involving the appropriate staff and other interested parties from the beginning of the process allows a more complete analysis to be made, promotes cooperation, improves relations both inside and outside TVA, results in the most appropriate options to be recommended, and concludes with support of all participants for the recommended options.

Zebra Mussel Control at Hydropower Facilities

Elba A. Dardeau, Jr.[1], and Tony Bivens[2]

Abstract

Zebra mussels were introduced into North American waters (Lake St. Clair, Michigan) in 1986 when a ship from a European freshwater port released its ballast water. These organisms have quickly spread from the Great Lakes to many other North American waterbodies. They are macrofoulers that quickly colonize new areas on many different types of natural and artificial substrates. Particularly vulnerable are power production facilities that withdraw large quantities of raw water to generate electricity. Infested waters pose a serious threat to hydropower components. The development and field testing of environmentally sound but effective zebra mussel control strategies has been initiated to determine which strategies are most suitable for the power production components at risk. A pilot study on the Cumberland River is discussed.

Introduction

The adult zebra mussel (*Dreissena polymorpha*) is usually less than 5 cm long and has characteristic zebralike stripes (Figure 1). It arrived in North America in 1986 probably as a result of a freighter releasing ballast water that had been taken on at a European port. Since that time they have quickly colonized many other North American waterbodies.

[1] Geologist, Environmental Laboratory, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS 39180

[2] Manager, Hydropower Operations, Operations Division, U.S. Army Engineer District, Nashville, Nashville, TN 37202

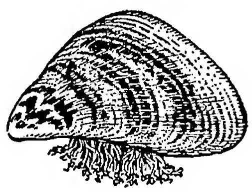


Figure 1. Zebra mussel (*Dreissena polymorpha*)

Ecology

In discussing the ecology of zebra mussels, Miller, Payne, and McMahon (1992) state that these organisms

> are found in freshwater lakes, embayments, and rivers. If temperature and water quality are appropriate, they tolerate velocities up to 2.0 m/sec (6.6 ft/sec). They are typically are found where water temperatures range from 0° to 25° C (32° to 77° F). They have been collected in shallow waters (less than 1 m or 3.3 ft), but maximum abundance usually occurs between 2 to 14 m (6.4 to 44.8 ft). Zebra mussels are clean water inhabitants and are usually found where dissolved oxygen is greater than 90 percent saturation. They are stressed in water with less than 40 to 50 percent saturation, and 100 percent mortality occurs if there is no dissolved oxygen. Zebra mussels, like all bivalves, require calcium to construct their shell. They will not be found in water with less than 10 mg/l dissolved calcium.

Zebra mussels attach themselves to various substrates. In addition to rocks, wood, aquatic vegetation, and the shells and exoskeletons of other exotic species, they have also managed to survive and often thrive on plastic, concrete, fiberglass, iron, and polyvinyl chloride. They form colony densities as large as hundreds of thousands per square meter. More importantly, they are able to form layers that are several centimeters thick that can add considerable mass to components of hydraulic structures and thus greatly reduce their efficiency or render them inoperable. Miller and Lei (in preparation) provide a method for calculating total mass of zebra mussels in or out of the water.

Life Cycle

Zebra mussels consist of dioecious individuals (both sexes), who release sperm and eggs into the water for external fertilization. Once fertilized, the embryo develops inside the egg within a required temperature of 12° to 24° C. It hatches as a free-swimming microscopic larva known as a veliger. The veligers can be transported by water currents and very easily settle on any of the many surfaces described above and, if the environment is favorable, they grow into an adult with byssal threads attached to hard surfaces, including shells and exoskeletons of other organisms (McMahon and Tsou 1990).

Authority

By means of the Nonindigenous Aquatic Nuisance Prevention and Control Act of 1990 (Congressional Record House, 27 October 1990), Congress has specified that the Assistant Secretary of the Army, Civil Works, will develop a program of research and technology development for the control of zebra mussels (*Dreissena polymorpha*). As a result of this legislation, the US Army Corps of Engineers has initiated a 4-year program managed at the US Army Engineer Waterways Experiment Station (WES), Vicksburg, MS, to develop environmentally sound control strategies for control of this nuisance aquatic species. The focus of the research is public facilities along US waterways and includes a) water intakes and outlet works, b) locks and dams, c) vessels and dredges, and d) hydropower and other electrical utilities. To accomplish the objectives of the research program, working groups were formed to prioritize components of concern and to identify control strategies that could be tested. This paper deals specifically with zebra mussel control at hydropower facilities, using Cheatham Dam on the Cumberland River as an example.

Working Group

A working group of experts knowledgeable on the subjects of hydropower facilities of zebra mussel biology and control was organized to identify components of a power production facility that were likely to be at risk from zebra mussel infestations and to determine the best environmentally acceptable strategies for dealing with these infestations. The group consists of Corps division and district employees involved in the operation of hydropower facilities, scientists and engineers from the WES and the US Army Construction Engineering Research Laboratories, Champaign, IL, plus representatives from

the private sector and other federal agencies that operate hydropower plants. Ontario Hydro has also been an active participant, willing to share its expertise and experiences in controlling zebra mussels at power production facilities. The working group, which has proved to be an ideal forum for information sharing and exchange of ideas, first met in January 1992 in Nashville and again in January 1993 in St. Louis. The plan is to reconvene during each succeeding January for at least the next 2 years. Members of the group also interact with the others in the power facility community and in the zebra mussel research community as a whole through participation in national and international conferences, such as this one, and by publishing findings of field and laboratory studies.

Components at Risk

Bivens, Dardeau, and Payne (1992) summarized the findings of the January 1992 working group meeting in a Zebra Mussel Research technical note. Facility components were prioritized in terms of their vulnerability to zebra mussel infestations, and control strategies were proposed for each component. As would be expected, the participants agreed that components that came into contact with or relied on raw water for their operation were in greatest jeopardy. In addition, instrumentation, external devices (e.g., fish ladders or trash racks), drains, imbedded piping, valves, and water supply lines were also determined to be at risk.

Zebra mussels easily adapt to new surroundings, and they are able to grow and rapidly reproduce under a broad range of environmental conditions. The components at risk must, therefore, be protected so that their functions are not markedly degraded. Method(s) chosen must be effective, yet environmentally sound, such that National Environmental Policy Act (NEPA) guidelines and other agency permit constraints can be satisfied for the locality of interest.

Monitoring

Ingram and Miller (1992) pointed out the necessity for a sound monitoring program to permit early detection. Monitoring devices range from very simple hand-held boxes, such as that described by Miller and Dye (1992), to more sophisticated veliger detection systems, such as those tested by Ontario Hydro (Payne and Lowther 1992). Ideally, any monitoring system at a hydropower facility should be in place prior to infestation. Once a site is

infested, however, monitoring devices can be used to control the timing and duration of treatment application.

Treatments

Many different types of treatments or control strategies have been tested or proposed. Chemical treatment and physical removal (e.g., scraping or high-pressure jetting combined with suctioning) have proved to be most promising. In addition, antifoulant coatings (whether thermal-sprayed, copper-based, zinc-rich, or nontoxic foul-release) also offer protection. Other control methods tested at hydropower installations include sonic waves, heat, and desiccation; these treatments have met with varying degrees of success, some being practical and others being costly or difficult to implement.

Of the chemical measures tested thus far, chlorine application appears to be very promising, especially when coupled with a sound monitoring program that can be used to determine when treatment should begin and end and what concentrations should be used. Ontario Hydro is conducting research on a number of different methods for zebra mussel control, but they have found chlorine application to be very reliable.[3] Chlorine is toxic to veligers and can be applied in legal concentrations that satisfy the requirements of NEPA and other environmental constraints. A protocol should be developed for any treatment program to ensure environmental compliance. Miller et al. (1992) provide the description of such a protocol that the US Army Engineer District, Nashville, has developed for management of its projects in the Cumberland River basin. The protocol enables the District to complete as much of its "environmental homework" as possible prior to the onset of zebra mussel infestations at its hydropower facilities.

Case Study - Cheatham Dam

Zebra mussels are in the lower reaches of the Cumberland River in Kentucky and Tennessee. Nashville District personnel are especially concerned because they have found adult organisms in lock chambers adjacent to hydroelectric plants at Barkley and Cheatham dams. No components of the District's hydropower facilities are

[3] Personal communications between the authors and Messrs. Paul Wianko and Dave Lowther and Ms. Renati Claudi, Ontario Hydro, Toronto, January - September 1992.

infested; however, the organisms are dangerously close, and immediate action was required. One facility that has been equipped with a chlorine injection system is Cheatham Dam (Cumberland RM 148.7).

The Cheatham Project was the sixth of nine hydropower projects built for the Nashville District. It was completed in 1960. Cheatham hydropower plant is of low-head design with three 20,000-horsepower vertical shaft kaplan (propeller type) turbines with a design operating head of 22 ft. Generators are three-phase, 60-cycle, 13.8-kv units rated at 13,333 kva and 60 rpm.

Excess heat is removed from the Cheatham turbine bearings, generator bearings, and generator windings by raw-water-cooled heat exchangers. Cooling water is drawn from the river, circulated through the system and then returned to the river. Much of the pipe supporting this operation is embedded in the concrete and has sharp bends, which only adds to the vulnerability of this facility to a zebra mussel infestation. Without suitable protection, the Cheatham hydropower plant could be completely nonfunctional in one reproductive season should fouling of the raw water cooling system occur.

After developing a zebra mussel infestation control strategy, which included water chemistry analyses, monitoring/detection systems, determination of facility components at risk, coordination and environmental documentation, Nashville District was ready to design and implement its controls. In spring 1992, the district developed a design for chlorine injection system to be installed on the raw water systems at all of their hydropower plants, among them, Cheatham. The principal system components are a variable-rate chemical injection pump, which feeds chlorine into the raw water system at its source; storage tanks sized to hold a 30-day supply of chlorine; an analyzer, which measures the concentration of chlorine in the discharge water and sends a signal to a program logic controller to automatically adjust the chemical feed pump and maintain the desired chlorine concentration; and a control panel with a strip chart recorder, which continuously records the discharge water chlorine concentration.

The chlorine injection system will be activated when veligers are detected. Initially the system will be activated to produce a total residual chlorine level at the raw water discharge point of 0.5 ppm on a continuous basis. Cheatham will, however, experiment with different concentration levels and injection intervals to determine the optimum protection that will involve the minimum use of chlorine.

With this installation at Cheatham, Nashville District has protected the most vulnerable site components, the raw water cooling systems. Even though other facility components may also be at risk, they are physically accessible and amenable to other control measures.

Conclusions

Zebra mussels pose a major threat to public facilities in the US, especially power production installations. Facilities that rely on single-pass water cooling systems are particularly at risk. These organisms are adaptable to a broad range of environmental conditions and are able to form colonies of very dense layers that can add considerable mass and hamper the operation of hydraulic structures. In the case of hydropower installations, such macrofouling could result in loss of efficiency or even down time, which translates into lost revenue.

Even though the zebra mussel problem is relatively new, and there are few natural enemies for these organisms, control measures are possible. Full advantage should be taken of the experience gained by Ontario Hydro and others in combatting this problem over the past several years. Still in the process of researching the efficacy of different control strategies, Ontario Hydro has found that chlorine application has been consistently reliable.

The Cheatham Dam case study was presented to illustrate the importance of having an in-place control system prior to the onset of infestation.

Recommendations

Existing technologies can be adapted to provide effective zebra mussel infestation controls. Future methods that are less environmentally damaging need to be developed.

Environmental concerns must be a part of planning for zebra mussel infestation controls. With thorough study and planning, electric power production can continue unaffected as zebra mussel infestations occur. Federal facilities that want to operate chlorine and other chemical injection systems will have to obtain National Pollutant Discharge Elimination System (NPDES) permits. The Environmental Assessment and the NPDES application process should be started as soon as possible.

Acknowledgement

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Erosion and Sediment Control Plans For Hydropower Projects

Kathleen Sherman[1]

Abstract

The Federal Energy Regulatory Commission (Commission) is responsible for processing applications for licensing nonfederal hydropower projects, issuing orders licensing such projects, and ensuring that licensed projects are operated safely and in compliance with the license order.

A case study is the 760-megawatt (MW) Rocky Mountain Pumped Storage Project. This project is located on Heath Creek in Big Texas Valley, in Floyd County, Georgia. The project was licensed in January 1977. Construction began in October 1978; as of July 1992, construction was close to 50 percent complete.

To comply with the requirements of the National Environmental Policy Act, an Environmental Impact Statement (EIS) was issued in May 1976, and a Supplement to the EIS was issued in June 1981 for the relocation of the lower reservoir main dam. The Licensee filed a soil sedimentation and erosion control plan pursuant to Article 32 of the license.

When preparing final plans and specifications for building the project, the licensee develops the actual construction sequence and grading plans and makes appropriate design changes in the erosion control plan.

The Commission's Regional Offices conduct periodic construction inspections to ensure that the project is being constructed in accordance with the plans and

[1] Soil Conservationist, Office of Hydropower Licensing, Federal Energy Regulatory Commission, Washington, D.C. 20426

specifications and that the contractor is providing a quality product. However, direct and continuous on-site surveillance by the Commission is not possible because of budget limitations. So, before construction begins, licensees are required to file a quality inspection program for approval by the Commission, which includes an inspection and monitoring program for erosion and sediment control measures.

Since actual site conditions can vary during the course of construction, modifications to the erosion and sediment control plan must be accomplished in the field as conditions dictate. There may be situations where erosion control measures are not working effectively or must be modified because of unforseen conditions. Due to the need to react to changing conditions as they arise, the quality control program allows modifications to be made in the field with the agreement of the Regional Offices.

The Rocky Mountain Project illustrates how the quality control program enabled implementation of the erosion control plan to be flexible when unexpected field conditions developed.

Project Description

The Rocky Mountain Pumped Storage Project is located on Heath Creek in Floyd County, Georgia, 10 miles northwest of the City of Rome (figure 1).

The project will consist of a 221-acre upper reservoir on top of Rocky Mountain (full pond elevation 1,392 feet), a lower reservoir on Heath Creek consisting of a 600-acre operation pool (full pond elevation 710.5 feet) and two auxiliary pools of 400 acres and 205 acres (full pond elevation 715 feet) located in Big Texas Valley, a tunnel and underground penstocks, a semi-outdoor powerhouse with 3 reversible 225-MW pump turbine generating units, and a 3-mile-long 230-kV transmission line (figure 1).

The project, upon completion, will encompass about 5,000 acres. Of the total, 1,426 acres will be inundated. The remainder will be used for project operation, recreation, and wildlife management (Federal Power Commission, 1976).

The project will operate for 8 hours daily, generating during the day and pumping the upper reservoir full at night. There is sufficient storage provided for the plant to operate for an additional 45-minutes during emergencies (Federal Power Commission, 1976).

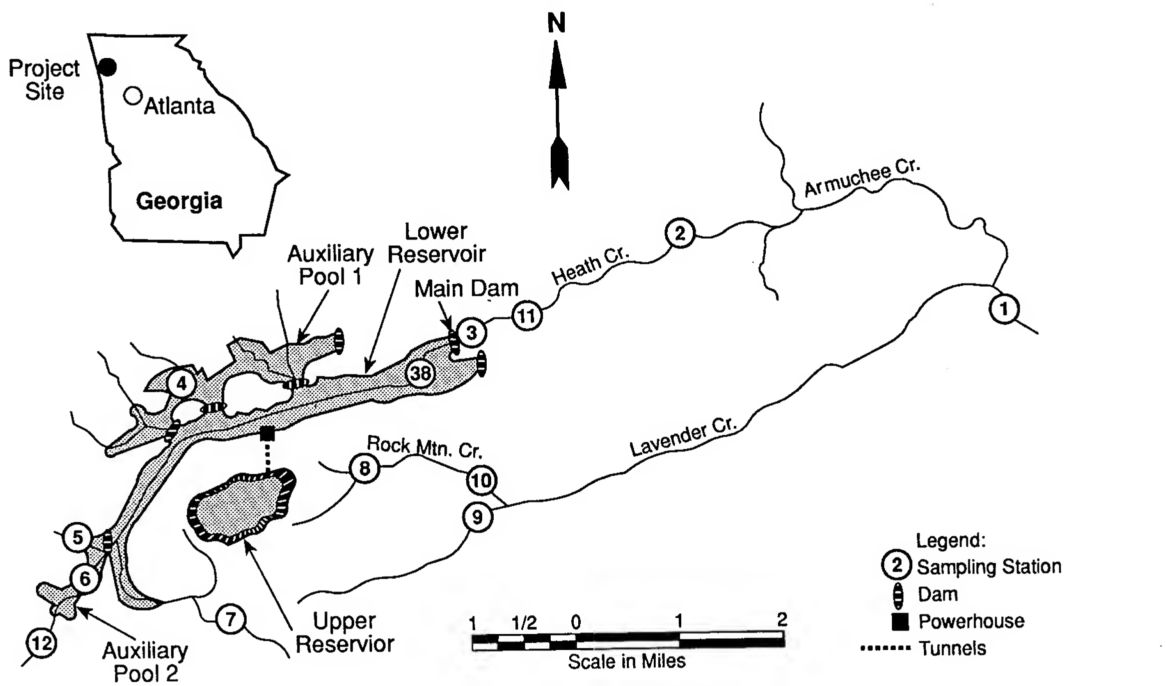

Figure 1. Features of the Rocky Mountain Pumped Storage Project and locations of water quality stations (Source: staff).

Reserve storage provisions allow operation to be independent of natural stream flow. Inflows from Heath Creek will be required only to make up losses due to evaporation and seepage. If reserve storage in the lower operating pool is not sufficient, water can be drawn from the auxiliary pools (Federal Power Commission, 1976).

Site Conditions

The project area is in a valley bounded by Simms Mountain on the north and Lavender Mountain on the south. The middle of the valley is occupied by Rocky Mountain, an isolated erosional remnant. This area physiographically, is situated within the Valley and Ridge province of the Appalachian Mountain chain (Oglethorpe Power Corporation, 1991).

The soil mantle throughout the project is generally thin, and outcrops of the underlying bedrock are numerous. At most exposures, the soils grade readily to deeply weathered and decomposed rock. Residual, alluvial, and colluvial deposits -- derived from the area's sedimentary sequence of sandstones, siltstones, and shales -- comprise most of the soils (Federal Power Commission, 1976).

Land-disturbing Activities

Construction procedures will require the excavation of about 4,500,000 cubic yards of earth material and about 5,200,000 cubic yards of rock material, with about 70 percent being used in construction of the dams. Construction activities involve the clearing of staging areas, development of a network of access roads, excavation of material from the tunnel, preparation of the damsite, and various other activities. Spoil areas are used to dispose of unwanted rock and soil. Over 1,000 acres will be cleared for reservoir construction (Federal Power Commission, 1976).

A 125-foot-wide clearing will be needed for the transmission line corridor. Erosion could be a serious problem, especially at water crossings or in areas of steep topography. Access roads to the upper reservoir site have space limitations with steep inclines, subject to rock falls and soil movement. Erosion, if not properly controlled, could become a serious problem due to the heavy, yearly rainfall. During construction of recreation facilities, some grading, excavation and clearing will be needed (Federal Power Commission, 1976).

Preliminary Erosion Control Measures

Prior to licensing, the Applicant proposed to make every reasonable effort to prevent erosion resulting from grading and excavation by the use of proper drainage design, contour barriers, and reseeding after construction (Federal Power Commission, 1976).

Preliminary plans for erosion control were prepared for the purpose of ensuring that there would be adequate erosion control during and after construction. This includes establishing grass, grading, and landscaping of disturbed areas. Spoil areas are being contained within reservoir sites, where possible. Unnecessary clearing is not allowed, and clearing of the lower reservoir site will take place just prior to filling to minimize erosion.

Settling ponds are being used near construction sites to catch drainage from these sites and allow suspended solids to settle out before water is released. Erosion caused by access road construction is being minimized by using existing roads where possible. Where new roads are needed, routes are selected for use as primary access roads after construction is completed.

Following construction, all road slopes, staging areas, spoil and borrow areas, and all areas not used for project operation will be grassed and tree seedlings planted to prevent erosion. The transmission line right-of-way will be planted in wildlife food and cover plants to minimize erosion on steep slopes.

License Requirements

The project license includes articles requiring the licensee to prepare a plan to control erosion and minimize sedimentation at the project in consultation with Georgia Department of Natural Resources (Article 32) and to monitor the effects of construction on surface waters -- including measuring turbidity and suspended sediments (Article 31) (Federal Power Commission, 1977).

Initial Erosion Control Plan

The initial erosion control plan was part of a pollution control plan -- filed in response to Article 32 of the project license (Georgia Power Company, 1978).

The primary considerations for the erosion control plan are (1) elimination of excess surface runoff crossing the disturbed areas, and (2) containment of sediments on disturbed areas or in sediment basins. The

sediment basins are used primarily to collect water from construction sites that is transported by gravity or by pumping. Sediment basins are constructed near each structure so that the length of transport can be held to a minimum. The majority of the sediment basins are planned to isolate them from sources of clean surface runoff to reduce the amount of water to be treated and to minimize the possibility of overflow during storms.

Vegetation is planned to be established on the sediment basin sites at the termination of their uses. In some cases these sites are expected to be regraded to conform with the natural topography before being vegetated. In most cases, a combination of sediment control practices is expected to be required in order to assure proper sediment control. In all areas the vegetation established will be consistent with the landscape plan required by Article 42 of the project license.

Activities that require the disturbance of natural slopes are isolated from upslope runoff by diversion dikes or ditches in order to minimize the surface runoff crossing the disturbed areas. The downslope perimeters of the disturbed areas are surrounded by sediment fences, hay bale barriers, brush barriers, etc. in order to keep the sediment on the disturbed area. Buffer zones have been left -- where practical -- to help remove sediment before surface runoff from a disturbed area reaches a stream or reservoir.

As construction in specific areas permits, interceptor dikes and vegetation will be established on the disturbed slopes to minimize erosion. Some areas are also graveled after grading to reduce erosion. Further, sediment control measures will be used during operation and maintenance of the project whenever ground-disturbing activities are unavoidable. Final disposal of sediments removed from sediment basins or from behind sediment barriers, will be by covering with topsoil and establishing permanent vegetation, burying, or by other suitable means.

Site-Specific Erosion Control Plan

A site-specific erosion and sedimentation plan was prepared after plans and specifications for project features were finalized. This plan divides the project site into six areas at a scale of 1 inch equals 200 feet (Oglethorpe Power Corporation, 1990a).

The site-specific plan also includes detail sheets that show (1) sediment ponds and outlet spillways, (2) sediment barriers including silt fence, hay bales, berms and dikes, and (3) rip rap headwall. Specifications for temporary seedings are also defined including species, seeding rates, and planting dates.

Plan drawings show locations of specific measures such as settling ponds, diversion ditches, filter berms (sediment traps), sediment barriers such as silt fence, rip rap, disturbed area stabilization with temporary seedings and permanent vegetation above the inundation level, and also show borrow and spoil areas.

A site narrative addresses stockpiling topsoil, minimizing clearing limits, limiting the exposure of bare slopes, seeding and mulching areas immediately after permanent works are completed, applying gravel to parking areas and roads, installing storm drain outlet protection devices, and installing diversions and dikes to divert sediment laden runoff into sediments basins and to protect cut and fill slopes.

Other items include: (1) diverting runoff from borrow areas to sediment control facilities; (2) discharging water pumped from structure excavations into sediment control ponds that are sufficiently large to provide adequate retention times for settlement of sediment; (3) using a series of temporary sediment settling ponds in natural drainage courses; and (4) using silt fences to intercept sediment during clearing operations and to intercept initial runoff, and along any perimeter requiring protection.

The site narrative also provides for runoff from equipment yard operations, drilling operations waste, and disposal of spoil material from excavations. The maintenance program provides for inspecting sediment and erosion control measures daily and repairing any damages observed by the end of that day, and addresses cleanout of sediment control structures.

Retention facilities and other erosion and siltation control devices must be installed prior to start of other construction and maintained until permanent ground cover is established or the site is inundated. The plan also notes that additional measures may be required and that all erosion control barriers must be maintained.

Standards and specifications for erosion and sediment control measures will conform to, and all work will be performed in accordance with, the "Manual for Erosion and Sediment Control in Georgia".

Quality Construction Inspection Program

Licensees are required to submit, prior to construction, a construction inspection program for Commission approval. Developers must include an inspection and monitoring program for erosion and sediment control measures as part of their construction quality control program. For large projects, a Board of Consultants periodically reviews both the design and construction activities at a project.

Developers are instructed to include in their monthly construction inspection reports a discussion of erosion control measures and their effectiveness. When appropriate, the report would include (1) a discussion of any instances where sediments or other construction discharges entered a stream, (2) the extent of the discharges, (3) an assessment of any damage to a stream, and (4) corrective actions taken, including measures to prevent further problems.

The Inspection Program for Construction Activities developed for the Rocky Mountain Project specifies daily inspections of all elements of on-going work by Construction Management staff. This includes environmental concerns as well as structural and other engineering features. For example, some of the elements of inspection specified to be emphasized during routine daily inspections are: (1) control of sediment; (2) dewatering of the excavation, treatment of water, and return of water to Heath Creek and its tributaries; (3) maintenance of stockpile and spoil areas; and (4) maintenance and use of access and haul roads (Oglethorpe Power Corporation, 1991b).

Modifications to Erosion Control Plan

When silt fence barriers were found to be ineffective in preventing sediment-laden runoff from large cleared areas from entering Heath Creek, additional measures were installed in 1991 in consultation with the Georgia Soil and Water Conservation Commission (Oglethorpe Power Corporation, 1993).

This included placing two large rock check dams in Heath Creek upstream of where the creek has been diverted around the construction area for the Main Dam. The rock check dams created two large sedimentation traps along Heath Creek. These ponds are about 30 acres and 40 acres in size. They are about one mile in distance within Heath Creek.

Interceptor swales were also installed along the base of lang or steep slopes. The swales were designed to capture water traveling at high velocity and routed to detention ponds to capture silt. Thousands of linear feet of interceptor swales were located adjacent to Heath Creek. Data on collected at water quality stations located upstream and downstream of the project site (figure 1) show similar levels of suspended sediment (Oglethorpe Power Corporation, 1992). This data indicates that the additional erosion and sediment control measures are effective.

Unexpected erosion also developed in the powerhouse backslope. A detailed investigation of the slopes found extensive deep gullying, local erosion holes, and local surficial slumping. A more formal plan was developed to correct this problem because of the potential for slope instability. The development and implementation of this plan was overseen by the Board of Consultants (Oglethorpe Power Corporation, 1990b).

The plan developed for the powerhouse backslope included backfilling scour holes with rockfill and impervious fill, regrading berms to redirect runoff, a network of concrete-lined ditches, drop inlet boxes, and grouted rip rap to route runoff from the backslope to the lower reservoir below, hydroseeding, and planting pine trees. The plan includes a drawing showing the locations of specific control measures as well as detail drawings showing plans and specifications for specific types of control measures (Oglethorpe Power Corporation, 1991a). Inspection reports by the Commission's Regional Office document that the backslope has been stabilized (Federal Energy Regulatory Commission, 1992).

CONCLUSION

The Rocky Mountain Project demonstrates that no matter how well conceived, the success of an erosion and sediment control plan depends on how well it is implemented. Control measures must be installed properly at the appropriate time and maintained, in order to function effectively. Further, the plan must be flexible enough to allow for modifications to be accomplished in the field as conditions dictate.

The Commission's process for developing an erosion and sediment control plan which includes inspections oversight -- together with the licensee's quality inspection program -- provides an approach that is workable and achieves the desired end result of containing sediment on-site and preventing it from reaching watercourses.

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Bank Erosion Resulting From Hydroelectric Operations

Michael J. Vecchio,[1] Alan W. Wells,[2]
Thomas L. Englert,[3] and David S. Battige[4]

Abstract

As part of a series of resource impact studies conducted to fulfill the requirements of the Federal Energy Regulatory Commission's hydroelectric license application, Consumers Power Company (Jackson, Michigan) retained Lawler, Matusky & Skelly Engineers to evaluate what impact hydroelectric operations have on bank erosion rates on the Au Sable, Manistee, and Muskegon rivers (Michigan, Lower Peninsula). The objective of the study was to determine whether peaking operations produce bank erosion rates significantly higher than rates associated with run-of-river (ROR) operations. Two methods were applied to develop a data base of bank erosion rates associated with ROR and peaking operations: one used historical and recent aerial photographs of the rivers; the other involved deriving bank erosion rates from pointbar progradation rates. Both methods led to estimates of long-term rates of bank erosion. The concept of unaffected and affected river reaches, defined as reaches upstream of all hydroelectric projects on a river and those downstream of projects, respectively, was employed to differentiate ROR erosion rates from those associated with peaking operations. Each estimate of bank erosion rate derived through

[1]Project Manager, Lawler, Matusky & Skelly Engineers, One Blue Hill Plaza, P.O. Box 1509, Pearl River, NY 10965

[2]Biometrician, Lawler, Matusky & Skelly Engineers, One Blue Hill Plaza, P.O. Box 1509, Pearl River, NY 10965

[3]Manager, Environmental Sciences & Modeling Group, Lawler, Matusky & Skelly Engineers, One Blue Hill Plaza, P.O. Box 1509, Pearl River, NY 10965

[4]Operations Engineer, Consumers Power Company, 1945 West Parnall Road, Jackson, MI 49201

photoanalysis was further qualified based on river hydraulic and morphologic characteristics. Analysis of covariance (ANCOVA) was used to (1) establish what effect mean flow rate, stream gradient, and bank height have on bank erosion rates and (2) filter out these effects in the final assessment of whether peaking operations produce bank erosion rates significantly higher than those produced by ROR operations. From these analyses it was concluded that, of six hydroelectric projects studied, historical peaking operations produced higher rates of bank erosion than ROR operations would have at three projects. The results were significant at the $\alpha=0.05$ level.

Introduction

Consumers Power Company owns 11 hydroelectric plants on three river systems in Michigan. As part of a relicensing effort, Lawler, Matusky & Skelly Engineers (LMS) was contracted to study how peaking and ROR hydro-electric plant operations affect bank erosion on these three rivers. The hydroelectric plants considered in the study were the Tippy and Hodenpyl projects on the Manistee River; the Foote, Alcona, and Mio projects on the Au Sable River; and the Croton project on the Muskegon River. A review of operation records showed that the plants under consideration here had operated in a peaking mode or a modified peaking mode, i.e., maintained a minimum flow, throughout much of their history. Consequently, bank ero-sion estimates derived for locations downstream of the plants and for the time period since dam closure were considered to reflect peaking opera-tions.

Two methods were applied to develop a data base of bank erosion rates associated with ROR and peaking operations: one used historical and recent aerial photographs of the rivers; the other involved deriving bank erosion rates from estimates of pointbar progradation rates. The bank erosion rates derived through photoanalyses were analyzed with a co-variance statistics package (ANCOVA) to identify which hydroelectric plants' peaking operations produced bank erosion rates significantly different from ROR rates. The results of the study are as follows:

- Bank erosion rates computed for the historical peaking condition downstream of the Tippy, Hodenpyl, and Foote projects are higher than those that would have resulted under ROR. The difference in rates is significant at the $\alpha = 0.05$ level.

- Bank erosion rates computed for the historical peaking operations of the Croton and Mio projects are lower than those that would have resulted under ROR, but the

differences are not statistically significant. Bank erosion rates computed for the historical peaking operation of the Alcona project are approximately equal to those that would have resulted under ROR.

Methods

Definition of ROR and Peaking Bank Erosion Rates

Photoanalytic and tree aging techniques were used to define rates of bank erosion associated with ROR and peaking operations. ROR bank erosion rates were derived from (1) photoanalysis of streambanks in unaffected river reaches (i.e., upstream of all hydroelectric projects on a given river), and (2) tree aging along pointbars located downstream of hydroelectric projects using trees whose ages predate upstream dam closure. Because aerial photographic records do not exist for dam pre-closure dates, ROR bank erosion rates for river reaches currently affected by dam operations were estimated solely by the tree aging method. Peaking bank erosion rates were defined as (1) rates estimated for river reaches downstream of a given project, using sets of post-closure aerial photographs; and (2) rates estimated by the tree aging method, using trees whose ages are less than the age of the upstream dam.

Stream Segment Classification

To facilitate the ANCOVA analysis of bank erosion rates derived through photoanalysis, characteristics thought to influence rates of bank erosion were defined for each study site. Stream gradient, median flow rate, bank height, and, where available, bank material type were assigned for each site where the bank erosion rate was estimated. Stream gradient and bank height data were obtained from U.S. Geological Survey (USGS) maps. Median flow rate data were obtained from flow frequency analyses compiled from 20 years of USGS gage data on the Au Sable, Manistee, and Muskegon rivers. Drainage basin ratios were applied to compute median flow rates along each river.

Bank material data for the rivers considered in this study were obtained from U.S. Department of Agriculture Soil Conservation Service documents. Additional information on bank material type was collected during the field surveys and from the Northwest Resource Conservation and Development Group.

Photoanalysis of Bank Erosion Rates

Affected and unaffected river reaches were analyzed for bank erosion using time-sequenced aerial photographs. For a given river reach the analysis involved (1) digitization of the river pattern from historical and recent photographs, (2) superposition and scale adjustment of the historical and recent river patterns using CADD graphics software programs, (3) measurement of the projected surface area of the eroded bank, and (4) computation of several rates associated with bank erosion. The various erosion-related parameters computed were:

- Maximum linear bank erosion rate (m/yr)
- Average linear bank erosion rate (m/yr)
- Volume of eroded sediment (m^3)
- Mass loading in metric tons per year (Mg/yr)
- Mass loading per river mile (Mg/yr-RM) of eroding bank

The mass loading rates of eroded sediment were calculated as the product of the volume of eroded sediment and the bulk sediment density divided by the time period separating the historical and recent sets of photos. Each mass loading value was divided by the number of river miles of eroding bank at that site to compute loading per river mile of eroding bank. When it was unavailable, a density value was assumed based on the range of densities observed in the region.

Pointbar Progradation: Tree Aging Method of Estimating Bank Erosion Rates

Three field surveys, one for each river, were conducted to review erosion conditions along the three rivers and to estimate bank erosion rates using the tree aging method. This method assumes that estimates of pointbar progradation approximate the rates at which the bank opposite the pointbar is eroding. The procedure for performing the tree aging method is summarized as follows:

1. Locate pointbar and approximate location of pointbar axis using standard field mapping techniques. Mark the axis with flagging tape.

2. Sample tree increments along the pointbar axis and record distance between trees and the distance from the shoreline to each tree sampled.

3. Age tree increment samples.

4. For affected river reaches, estimate the post-closure bank erosion rate as the distance (along the pointbar axis) from the water's edge to a tree of the same age as the dam divided by the age of the dam. This rate represents the maximum linear bank erosion rate associated with peaking operations at that site. Pre-closure bank erosion rates in affected river reaches are estimated in a similar manner except that the distance measurements are made relative to a tree of the same age as the dam.

Two steps were taken to increase the accuracy of the estimates of bank erosion rates: (1) pioneer tree species such as aspen, ash, cottonwood, willow, and white cedar were selected, and (2) tree ages derived from increment samples were corrected for the time required to reach breast height, the approximate point at which the samples were taken.

Analysis of Covariance

ANCOVA was performed on the data sets for unaffected and affected bank erosion rates derived through photoanalysis for each of the six projects. The unaffected data set used was a combination of all estimates of unaffected bank erosion rates derived through photoanalysis, e.g., upstream control reaches. The affected data sets were project specific and included only those estimates of bank erosion rates derived for a given project using the photoanalysis method. For each project ANCOVA computes unaffected and affected bank erosion rates using control and affected multiple regression equations* and stream gradient, median flow rate, and bank height conditions found in the control and affected river reaches. The analysis then corrects for inherent differences between the affected and unaffected river reaches, such as differences in flow, stream gradient, and bank height, by adjusting all observations to the grand mean before testing. The ANCOVA model was:

$$IA_{ij} = \mu + L_i + SL_i + Q_i + BH_i + \varepsilon_{ij} \tag{1}$$

where:

IA_{ij} = erosion rate at i^{th} location (m/yr)
μ = overall mean erosion rate (m/yr)
i = number of test locations, 1,2...k

*Due to limited bank material information, this variable was dropped from the analysis.

j = number of observations, 1,2,...n
L_i = effect of i^{th} location
SL_i = effect of stream gradient (covariate)
Q_i = effect of streamflow (covariate)
BH_i = effect of bank height (covariate)
ϵ_{ij} = residual error

Locations were defined as erosion sites upstream of all hydroelectric projects on a river (control) and erosion sites downstream of a given project (affected). For the three river systems the following locations were used.

River	Control Location	Affected Location
Muskegon	Above Rogers	Below Croton
Manistee	Above Hodenpyl	Below Hodenpyl and Tippy
AuSable	Above Mio	Below Mio, Alcona, and Foote

The statistical ANCOVA results presented were based on pooled control stations from all three rivers.

ANCOVA was conducted using the SAS GLM procedure (SAS 1985) with Type III sum of squares. The Type III model enters all variables into Equation 1 in a simultaneous rather than stepwise manner. All variables were $\log_e$ transformed because prior inspection had indicated that the variance for each variable tended to increase with its mean.

The effect of location (L_i) is the one of interest. A significant F-test, i.e., $P \leq 0.05$, was taken to indicate that the difference between at least one pair of locations was too great to have arisen by chance alone. Where significant location effects were detected, Dunnett's test (Steel and Torrie 1960) was used to isolate the location or locations that were significantly different from the control region.

Results

The results of the bank erosion analyses performed for affected and unaffected river reaches on the Manistee, Au Sable, and Muskegon rivers were used to (1) assess the utility of the photoanalytic and tree aging techniques and (2) compare rates of bank erosion as affected by peaking and ROR hydroelectric operations.

Method Comparison: Estimating Rates of Bank Erosion

Two methods to estimate bank erosion rates were used in this study to allow for verification of values derived from the aerial photoanalysis. A comparison of erosion rate estimates derived by using both methods is shown in Figure 1. For sites where erosion rate estimates were derived by the tree aging technique, photoanalysis was applied and these estimates are plotted against each other. Analyses of 10 affected streambank erosion sites and five unaffected sites are plotted together. Figure 1 demonstrates that, in both affected and unaffected river reaches, estimates of bank erosion rates predicted by tree aging are generally lower than those predicted by photoanalysis. This may be due to the erosion of pointbars over time or, in the case of post-closure rates, to limited sediment availability due to the behavoir of the dams as sediment traps. Both these effects would artificially lower estimates of bank erosion rates derived through tree aging. These comparisons indicate that basing the bank erosion analysis on comparisons of peaking and ROR erosion rates derived through photoanalysis is the more conservative approach, i.e., this method generates higher estimates of bank erosion rate. This effect was observed at both affected and unaffected sites and, consequently, should not bias the comparison of bank erosion rates attributable to ROR and peaking operations.

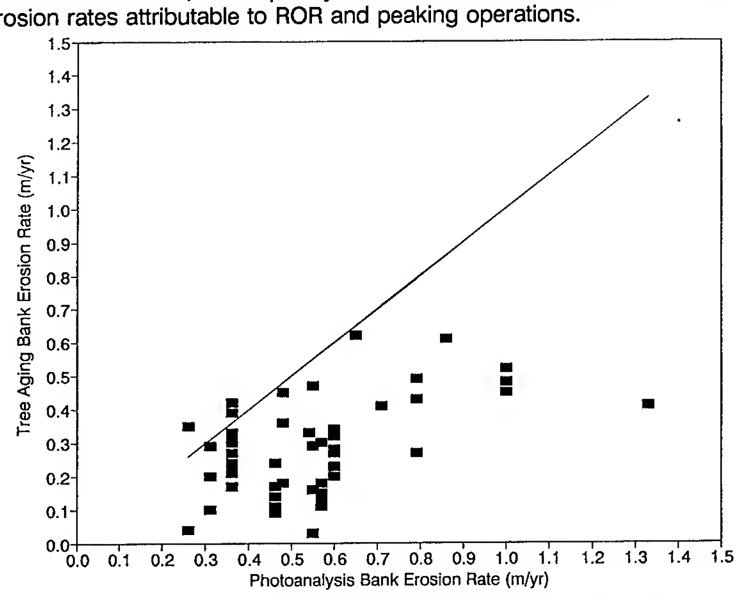

FIGURE 1. - Comparison of Bank Erosion Rates Derived From Photoanalysis and Tree Aging

Comparison of Peaking and ROR Bank Erosion Rates Derived through Tree Aging

Comparisons of peaking and ROR bank erosion rates derived through tree aging were made for the Foote and Hodenpyl projects. In this case, peaking bank erosion rates are those derived from the aging of trees representative of the post-closure period for a given project; ROR rates are those derived from the aging of trees representative of the pre-closure period. Of the six projects studied using the tree aging technique, only the surveys downstream of Foote and Hodenpyl resulted in a data base sufficient for comparing ROR and peaking bank erosion rates. The early closure date of Croton Dam (1908), coupled with extensive logging activity in the area, has caused a scarcity of trees older than the dam. Hence, pre-closure bank erosion rates could not be obtained at any of the sites studied below Croton. Bank erosion downstream of Alcona and Mio was extremely rare. Data collection was therefore limited and prevented the comparison of ROR and peaking rates.

Figures 2 and 3 are plots of pre-closure and post-closure bank erosion rates for the Hodenpyl and Foote projects, respectively. The 1:1 correlation line represents equal pre- and post-closure rates. All estimates of bank erosion rates derived during the surveys are shown on these plots. For example, for site HD1 downstream of Hodenpyl the tree aging technique produced two pre-closure estimates (0.31 and 0.08 m/yr) and one post-closure estimate (0.41 m/yr). Thus, the 0.41 m/yr post-closure (peaking) rate was plotted for both of the pre-closure rates. The results are more conclusive for Foote than for Hodenpyl. For the former, the post-closure rates consistently fall above the 1:1 correlation line, indicating post-closure (peaking) rates in excess of pre-closure (ROR) rates. However, downstream of Hodenpyl post-closure rates exceed pre-closure rates at low values of pre-closure rates but are less than pre-closure rates in the higher range.

Analysis of Covariance

The statistical results of ANCOVA are summarized in Table 1. The analysis indicates that the bank erosion rates computed for the peaking condition downstream of the Tippy, Hodenpyl, and Foote projects are higher than those that would have resulted under ROR and that the differences are significant at the $\alpha = 0.05$ level. The differences in bank erosion rates downstream of the Mio, Alcona, and Croton projects were statistically insignificant.

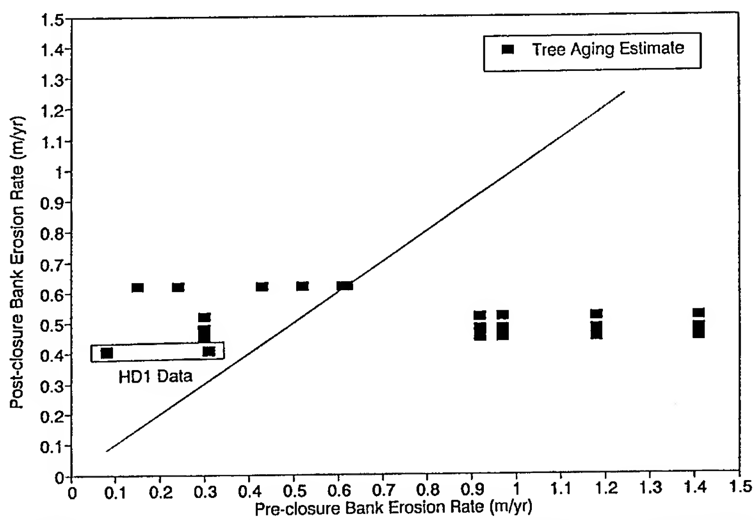

FIGURE 2. - Comparison of Pre- and Post-Closure Bank Erosion Rates Downstream of Hodenpyl Hydroelectric Project

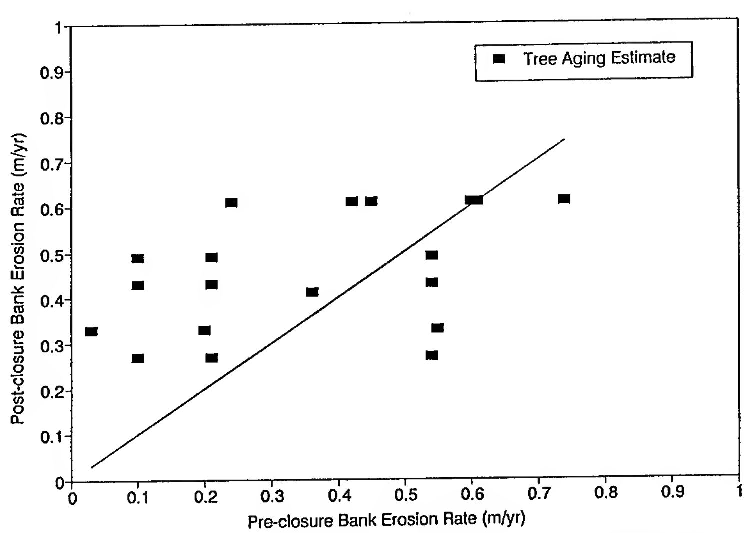

FIGURE 3. - Comparison of Pre- and Post-Closure Bank Erosion Rates Downstream of Foote Hydroelectric Project

TABLE 1. - Summary of Covariance Analyses

	Control Bank Erosion Rate (m/yr)			Affected Bank Erosion Rate (m/yr)			
Project	Low	X	High	Low	X	High	SD
Tippy	0.15	0.19	0.24	0.20	0.25	0.31	Yes
Hodenpyl	0.15	0.19	0.24	0.26	0.33	0.41	Yes
Croton	0.14	0.18	0.23	0.09	0.12	0.15	No
Mio	0.14	0.18	0.22	0.14	0.16	0.21	No
Alcona	0.14	0.18	0.22	0.15	0.18	0.23	No
Foote	0.14	0.18	0.22	0.21	0.26	0.32	Yes

X - mean value
Low - X - mean square error
High - X + mean square error
SD - statistically significant difference

Conclusions

Two methods of estimating bank erosion rates associated with ROR and peaking operations were applied in this study. The tree aging technique yielded a data base of sufficient size for evaluation of the Foote and Hodenpyl projects. Data collected downstream of the Foote project showed that peaking produced higher bank erosion rates than ROR operations would have produced. No consistent trend in the data collected downstream of the Hodenpyl project was observed; consequently, the tree aging analysis for this project was inconclusive. The ANCOVA based on results of photoanalysis, considered to be the most comprehensive, unbiased method applied in this study, identified the Hodenpyl, Tippy, and Foote projects as locations where peaking operations produced bank erosion rates that were higher than ROR operations would have produced. The differences in peaking and ROR rates were significant at the $\alpha=0.05$ level.

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The Endangered Species Act:
Unlawful Takings And An Incidental Take Permit

Sam Kalen[1/]

I. Abstract

The Endangered Species Act has become of increasing concern to many owners and operators of irrigation and water/power facilities. This paper briefly summarizes the Act, focusing primarily on the prohibition against "taking" an endangered or threatened species. This prohibition has been interpreted as encompassing a variety of private and governmental activities, including activities that affect entities dependent upon water management decisions.

This paper suggests that the Act's provision that authorizes granting incidental take permits is one potential solution for those entities concerned with the breadth of the "take" prohibition. An incidental take permit, and an accompanying habitat conservation plan, can offer certainty to project owners and developers, while at the same time provide an effective means for achieving the protection intended by the Act. This paper, consequently, briefly describes the incidental take permit process as well as the pros and cons of seeking such a permit.

II. Introduction

Congress passed the Endangered Species Act of 1973 ("ESA") in response to mounting concern over the extinction of fish, wildlife, and plants in the United

1/ Mr. Kalen is an Associate with Van Ness, Feldman & Curtis, A Professional Corporation, 1050 Thomas Jefferson Street, N.W., Washington, D.C. 20007. The views expressed in this article are solely those of the author and do not necessarily reflect the views of the firm or of the firm's clients. The author would like to thank Paul Turner for his invaluable assistance.

States. The Act primarily imposes procedural and substantive requirements on certain Federal actions and proscribes certain activity engaged in by private parties as well as governmental entities. Since its adoption and until the last few years, the ESA has been overshadowed by other Federal environmental laws and, with the exception of a few well-publicized cases, rarely has been perceived as a significant obstacle to ordinary project development.

Yet, during the past few years, the ESA has received wider attention, accompanied by the heightened focus on watershed management planning and protection of biodiversity. The two agencies primarily entrusted with administering the Act, the United States Fish & Wildlife Service ("USFWS" or "Service") for terrestrial species and the National Marine Fisheries Service ("NMFS" or "Service") for marine species, have more actively enforced the ESA's mandates; and through citizen suits, private parties have forced the agencies' hands as well as sought compliance with the ESA from third parties. Furthermore, Congress recently sought to ensure greater emphasis on wildlife protection when it passed the Reclamation Projects and Adjustment Act of 1992. And with the debate over the ESA now raging before Congress, the import of the Act cannot be understated.

III. Overview of the Endangered Species Act

The ESA is not a Federal permitting statute, such as the Federal Power Act or section 404 of the Clean Water Act. Instead, the Act, in its broadest terms, establishes (a) a process for formally listing endangered and threatened species and for designating their critical habitat, (b) a procedure for developing recovery plans for those listed species, and (c) a prohibition against Federal actions that are likely to jeopardize the continued existence of any endangered or threatened species, or its critical habitat, and a proscription against private or public actions that harm an endangered or threatened species.

Two of the primary provisions of the ESA are §§ 7 and 9. Section 7 governs Federal agency actions. If a Federal action is likely to affect a listed species, the ESA and the implementing regulations require that each Federal agency shall, in consultation with the USFWS or NMFS, insure that any action authorized, funded or carried out by that agency is not likely to jeopardize the continued existence of any listed species or result in the destruction or adverse modification of a critical habitat. An action jeopardizes the continued existence of a species if it "reasonably would be expected, directly or indirectly, to reduce appreciably the likelihood of both the survival and recovery of a listed species in the wild by reducing the reproduction, numbers, or distribution of that species."

Section 9 of the ESA prohibits any person, including governmental entities, from "taking" an endangered or threatened species, with certain limited

exceptions. Under existing interpretations, a variety of activities involving resource development can effectively "take" a listed wildlife species. The term "taking" is interpreted broadly to include "harass, harm, pursue, hunt, shoot, wound, kill, trap, capture or collect, or attempt to engage in any such conduct." An action "harasses" a listed wildlife species if it creates the likelihood of injury through disruption of normal behavior patterns, and an action "harms" such species if it kills or injures a species or adversely affects its habitat in a certain manner.

IV. Implications of the Endangered Species Act

The definition of harm is expansive enough to embrace habitat modification that adversely affects the recovery of listed species. For example, the State of Hawaii's practice of allowing feral goats and sheep to remain on a game management area and eat the leaves, stems, sprouts and seedlings of the mamane tree was challenged as a "taking" of the endangered Palila bird, which depends almost solely upon mamane trees for their habitat. In the most recent decision involving the effort to protect the Palila bird, a court held that the definition of harm includes habitat modification which threatens the extinction of the species in the future. The court also left open the possibility of finding a taking when there is habitat modification that adversely affects the recovery of a listed species. A different court reached a similar conclusion in a case involving forest management practices affecting the red-cockaded woodpecker. There, a court enjoined the United States Forest Service from authorizing the use of even-aged (clear-cutting) management practices in Texas wilderness areas. According to the court, such practices destroyed habitat for the woodpecker and led to a decline in the population of the species, which constituted an unlawful "taking."

Not surprisingly, therefore, the ESA recently has become a significant issue for many owners and operators of irrigation and water/power facilities. The number of circumstances are increasing where application of the Act may impact water management decisions (i.e., operational plans) and allocation. These circumstances, as well as a growing body of literature, highlight the need to understand and respond effectively to potential impacts to endangered and threatened species. One leading water law scholar, for instance, asserted in 1985 that "[t]he United States has a strong endangered species policy that potentially conflicts with state water diversion and impoundment projects," with the Act "effectively creat[ing] de facto regulatory water rights." The Columbia and Snake rivers are currently two noted examples where the debate has focused on the ESA. And, at least one author commenting on the Columbia River Basin suggests that streamflow depletions, which leave insufficient water to maintain habitat, should be a taking under § 9 of the Act.

Two recent disputes highlight the role of the ESA. In the first of these, NMFS sought to enjoin Glenn-Colusa Irrigation District ("GCID") from operating its pumping facility at Hamilton City, California, claiming that operation of the facility resulted in an illegal "taking" of the Sacramento River winter-run chinook salmon. GCID had been in a longstanding dispute over the appropriate solution for minimizing fish mortality at its facility. After GCID applied to the U.S. Corps of Engineers ("Corps") for a section 404 permit under the Clean Water Act, the Corps consulted with the NMFS pursuant to the ESA. In its biological opinion, NMFS determined that approving the 404 permit was likely to jeopardize the winter-run salmon.

In that opinion, however, NMFS stated that the jeopardy determination could be avoided if a new, efficient--but extremely costly--fish screen were constructed at GCID's facilities. The opinion also contained an incidental take statement addressing the undisputed "takings" of the winter-run salmon, concluding that the 404 permit could be issued if GCID installed an effective new fish screen. The court indicated that NMFS notified GCID that without an incidental take permit or unless GCID complied with the incidental take statement in the biological opinion, GCID would be liable for incidental takes under the ESA. The court issued a permanent injunction, indicating that it would not modify the injunction until GCID reported whether it would comply with or challenge the incidental take statement or apply for an incidental take permit.

In another instance, a district court held that the USFWS violated the ESA when it failed to develop and implement recovery plan(s) for endangered and threatened species inhabiting the Edwards Aquifer and the San Marcos and Comal Springs in Texas. Plaintiffs argued that the obligation to develop recovery plans included requiring that USFWS develop springflow levels for the Texas springs, in order to avoid violating the Act by either "taking" any species or jeopardizing the continued existence of any endangered or threatened species. According to plaintiffs, judicial action was necessary "to prevent takings and ensure the conservation and survival of the endangered species in the Edwards Aquifer and Comal and San Marcos Springs . . . . with the barest minimum of expense and disruption, and to preserve the maximum possible opportunity for meaningful state, local and federal action."

The Edwards Aquifer is an underground conduit, which is 175 mile long, spans some 3,600 square miles, and underlies the San Antonio and Gaudalupe River Basins. Water discharges from the Edwards through springs, primarily the Comal Springs and the San Marcos Springs. Water also is removed from the aquifer by pumpers. The Edwards Aquifer serves as a major water source for over a million residents and supplies the needs of municipalities, irrigated agriculture, industry, domestic and livestock uses and several military

installations. Edwards and the two springs further provide habitat for endangered and threatened species.

The court found that reductions in the springflow and water level of the aquifer have harmed those species and threatened their continued existence. During the past decade, however, USFWS allegedly neither implemented its recovery plan for San Marcos nor developed a plan for Comal Springs, as well as "never identified the necessary springflow requirements of the species." Plaintiffs claimed that this failure by USFWS violated the agency's obligation under the Act to develop and implement a recovery plan for the protected species, and that it also violated § 9 of the Act by allowing the "taking" of endangered or threatened species.

The court generally agreed with the plaintiffs, holding that the ESA requires that USFWS develop and implement recovery plan(s) for the listed species. According to the court, moreover, USFWS caused or allowed takings to occur, in violation of § 9, and the failure to develop and implement the recovery plan(s) increased the risk of jeopardy to the continued existence of the species. The court warned that "[a]lmost four decades of negotiations among the affected parties have failed to yield a resolution of the disputes regarding the proper management of the Edwards. *Continuing failure risks Federal intervention to protect endangered species.*"

The court, in part, ordered that USFWS, within 45 days, use its "best professional judgment" to determine the minimum springflow levels necessary to avoid taking or jeopardizing the fountain darter in the Comal and San Marcos Springs and to determine at what level "damage to or destruction of or adverse modification of the critical habitat of the Texas wildlife begin to occur." For the period before these minimums are established, the court established certain springflow requirements, which could be modified by USFWS at any time based on available information and its best professional judgment.

Additionally, the court strongly urged that the Texas Legislature adopt a plan for regulating withdrawals from the Edwards Aquifer in order to "avoid jeopardy to, and destruction or adverse modification of critical habitat of, any listed species." The court also directed that all entities pumping water from the Edwards Aquifer be given a copy of the court's decision and take appropriate action in the event Texas fails or refuses to regulate withdrawals "to the extent necessary to avoid unlawful takings of listed wildlife species. . ."

In both the Edwards Aquifer and GCID disputes, the solution, to date, appears to lie in the ESA's emerging flexibility in allowing what are termed "incidental takes" of listed species, provided that those "takes" do not risk jeopardizing any listed species. In response to the GCID decision, the parties have entered into a joint stipulation agreement, with GCID subsequently filing

an application (that is still pending) for an incidental take permit. In the Edwards Aquifer decision, the court noted its belief that adverse economic consequences would only result if the State and local authorities and regional water users fail to implement pumping controls and reduce dependence on the Edwards Aquifer "before the 'blunt axes' of Federal intervention under ESA §§ 7 and 9 fall." The court envisioned that one non-disruptive solution would be for the State to develop a plan for pumping controls and seek a § 10(a) incidental take permit, and "[t]he sooner Texas adopts an adequate plan and obtains a § 10(a) permit, the better."

V. Incidental Takings Under the Endangered Species Act

The ESA authorizes incidental takes to occur in two circumstances: first, pursuant to a Federal agency's consultation with USFWS/NMFS for an action that is approved, funded or carried out by an agency; or second, in connection with an application by a non-federal party for an incidental take permit. In 1982, Congress amended the ESA to authorize the issuance of a permit for incidental takings of endangered species. This amendment responded to a prolonged dispute over development at the San Bruno Mountain in northern California, an area that USFWS intended to designate as critical habitat for the listed mission blue butterfly and another proposed species. The result was a multi-party negotiated habitat conservation plan, through which private funding would be available for habitat acquisition and management. The 1982 amendments sanctioned such a plan.

The Act as amended allows private parties to "take" listed species if they have received and complied with the terms and conditions of an incidental take permit. Pursuant to section 10(a)(1)(B) of the Act, the Secretary "may permit, under such terms and conditions as he shall prescribe--any taking otherwise prohibited by section 9(a)(1)(B) if such taking is incidental to, and not the purpose of, the carrying out of an otherwise lawful activity." The conference committee explained that this provision created a procedure "whereby those persons whose actions may affect endangered or threatened species may receive permits for the incidental taking of such species, provided the action will not jeopardize the continued existence of the species. This provision addresses the concerns of private landowners who are faced with having otherwise lawful actions not requiring Federal permits prevented by section 9 prohibitions against taking."

The Act also allows Federal agency actions to go forward even if they might harm listed species, if they do not risk jeopardizing any species and the activity is consistent with an incidental take statement. Pursuant to section 7(b)(4) of the ESA, an incidental taking may be allowed in conjunction with a Federal agency action. If a Federal agency determines that an action or authorization of certain activity could result in an incidental taking, that agency

must obtain from the Service a written statement approving the activity. "This 'incidental take statement' operates to exempt the Federal agency and any permit or license applicant involved from the section 9 'taking' prohibitions under the Act if the subsequent implementation of the action is consistent with the terms and conditions of the incidental take statement."

Federal regulations prescribe the procedures for submitting an application for an incidental take permit. The application can seek authorization for a single transaction, a series of transactions or a number of activities over a specific time period. For example, when an applicant seeks an incidental take permit from USFWS, it must submit with the application a complete description of the activity sought to be authorized, the species sought to be covered by the permit and a conservation plan (a habitat conservation plan or "HCP"). The conservation plan must describe the likely impact of the taking, those measures the applicant will employ to minimize or mitigate any such impacts and any procedures for resolving problems posed by unforseen circumstances. The conservation plan also must identify and analyze alternatives to the incidental taking and discusses why they are not being utilized, as well as include any such other measures that the Service may require as "necessary or appropriate for purposes of the plan." Prior to formally seeking the permit, applicants are advised that they may discuss with the Service any possible measures and conservation plans. The ESA requires that notice and opportunity for public comment be afforded to all interested parties before the issuance of any permit--an action that is also subject to the procedures of the National Environmental Policy Act.

Before issuing any permit, the Service must make certain findings. It must initially determine that any authorized activity is not likely to jeopardize the continued existence of any listed species or result in the destruction or adverse modification of any critical habitat, and that the taking will be "incidental to, and not the purpose of, the carrying out of an otherwise lawful activity." The Service must also conclude that:

* the applicant will, to the maximum extent practicable, minimize and mitigate the impacts of such taking;

* the applicant will ensure that adequate funding for the plan will be provided;

* the taking will not appreciably reduce the likelihood of the survival and recovery of the species in the wild;

* the measures, if any, required by the Service as being necessary or appropriate for purposes of the plan will be met; and

* that the Service has received such other assurances as it may require that the plan will be implemented.

USFWS, for instance, also considers the "anticipated duration and geographic scope" of the proposed activities, as well as the "amount of listed species habitat that is involved and the degree to which listed species and their habitats are affected." Furthermore, any permits must contain terms and conditions necessary or appropriate to effectuate the purposes of the HCP, including monitoring and reporting requirements. A permit will be revoked if its terms and conditions are violated.

Incidental take permits arguably offer something for everybody. On the one hand, private parties can be reasonably assured that a range of activities in an area will not violate the Act. They also can reasonably expect that no further mitigation requirements will be imposed in the future. On the other hand, HCP's can provide one mechanism for obtaining effective wildlife protection, by including such elements as acquisition of land for wildlife purposes as well as the necessary funding for managing that land. Additionally, by approving a HCP, the Service can attempt to manage an ecological unit, potentially protecting diversity of habitat through broad land management programs rather than through the singular and often isolated focus of a lawsuit or proposed agency action.

Of course, developing a HCP that is acceptable to the Service and possibly other interested parties admittedly can be a difficult task. It can take considerable time, effort and money before an applicant receives an incidental take permit, particularly in those instances involving multi-party negotiations. An applicant may have to fund biological studies, engage in lengthy negotiations, respond to comments by interested persons, and possibly agree to on-site and off-site mitigation, including the acquisition of land for wildlife purposes and funding for management of that land. And the applicant must be satisfied that the incidental take permit will freeze in place the bar to any "taking" claim for authorized activities within a specific geographic area over a certain time period.

VI. Conclusion

While the debate in Congress continues over the reauthorization of the ESA, and as the public becomes mobilized, whether to urge for maintaining the status quo or to press for reforming the Act, persons or entities whose projects could implicate § 9 of the Act might explore the option of seeking an incidental take permit. The development of an acceptable HCP, in conjunction with an incidental take permit, offers a measure of certainty to project developers and operators, while at the same time provides a unique mechanism for achieving meaningful habitat and ecosystem protection for wildlife. The challenge, of course, is developing a conservation plan under highly visible circumstances, such as the Edwards Aquifer dispute, that satisfies a myriad of interests.

Relicensing The Ozark Beach Hydroelectric Project

Robert D. Wood[1]
William C. Howell[2]

Abstract

In 1991, 157 applications for new hydroelectric licenses were filed with the Federal Energy Regulatory Commission (FERC). The Ozark Beach Hydroelectric Project was the first of the 157 applicants to receive a new license. This paper describes the planning, teamwork, aggressive consultations, and attention to detail that enabled the Empire District Electric Company (Empire) to receive a new license in just over seven months after their filing date.

Introduction

The Ozark Beach Hydroelectric Project, owned and operated by the Empire District Electric Company, is located on the White River in southwestern Missouri. The project was one of the largest hydroelectric generating plants in the United States when it began operation in 1913. In 1931, the 25 Hz horizontal Leffel units were replaced with four 60 Hz vertical Francis units, which currently produce approximately 16 megawatts of power. Immediately downstream, the construction of the US Army Corps of Engineer's Bull Shoals project in the early 1950s prompted the construction of protective works at the Ozark Beach Project to mitigate the potential increase in tailwater elevation. Other than these modifications, the project appears much as it did in 1931.

In the late 1950s, the US Army Corps of Engineers constructed the Table Rock project approximately 23 river miles upstream of the Ozark

[1]Project Engineer, Black & Veatch, P.O. Box 8405, Kansas City, Missouri 64114.

[2]Superintendent of Production, Southern Division, Empire District Electric Company, P.O. Box 127, Joplin, Missouri 64802.

Beach Project. The Table Rock project was authorized for production of hydroelectric power and flood protection. Prior to construction of the Table Rock Project, the Ozark Beach project reservoir, Lake Taneycomo, was populated by a warm water fishery. However, cold water discharges from Table Rock Lake significantly reduced temperatures in Lake Taneycomo. Anticipating this temperature change, the Missouri Department of Conservation constructed the Shepherd of the Hills Trout Hatchery just below the Table Rock Dam. Today, Lake Taneycomo is Missouri's most popular trout fishery.

Because Empire's original FERC license had an August 1993 expiration date, they initiated the effort to relicense their project in early 1988. Although the Electric Consumers Protection Act (ECPA) was approved by Congress in 1986, regulations implementing the act would not be issued until June 1989. This lack of specific direction from the FERC, coupled with the scarcity of other relicensing applicants, assured Empire that they would be sailing in relatively uncharted waters in relicensing their project.

Project Approach

Relicensing a major hydroelectric project is a demanding process requiring expertise in many different disciplines. The relicensing process can be managed entirely by the potential applicant or by a licensing consultant. Most applicants lack the necessary staff needed and must retain consultants to prepare part or all of the application.

Project team. In early 1988, Empire retained Black & Veatch, a Kansas City, Missouri, based engineering firm, to provide engineering licensing, environmental, and project management support for relicensing the Ozark Beach project. Empire wanted to have hands-on participation in the project, and would produce portions of the document and provide land management, hydro operations, and legal support.

Project approach. In response to ECPA, Black & Veatch developed an integrated licensing approach for managing the tasks associated with the preparation of the license application. This approach outlines the process which enables the licensee to project resource requirements and costs, monitor overall progress, and make timely and informed decisions. ECPA related regulations had yet to be written, so Black & Veatch developed project tasks from the ECPA itself. Each task outlined the work to be performed and identified the responsible party (Black & Veatch or Empire). The tasks were interfaced using Critical Path Method scheduling techniques to ensure timely completion of the license application. After

FERC published its final rules in June 1989, Black & Veatch revised task descriptions, added tasks as required, and checked for schedule effects.

Although Black & Veatch would manage the daily details regarding the application production, Empire was closely involved with the planning and decision-making process. This partnership proved effective. Empire's attitude toward active project participation and civic responsibility complemented Black & Veatch's style of proactive management and attention to detail.

The FERC Consultation Process

ECPA considerably enhanced the role of resource agencies in the relicensing process. The ECPA also stipulated that an applicant must submit detailed information on such issues as resource utilization, need for the power generated, and power conservation. While this information (which would ultimately become Exhibit H of the license application) proved to be time consuming to produce, the consultation requirements and the enhanced role of resource agencies were the greatest concerns of the hydroelectric community.

Relicensing a FERC project requires preapplication consultations with resource agencies and the public. The consultations consist of three stages. The first stage begins when a potential applicant provides information describing the project and its setting, any proposed changes to the project, and any proposed environmental study plans. A joint meeting with pertinent agencies must be held to discuss data and needed studies.

During the second stage, the potential applicant conducts studies and collects information requested by the agencies. This information is submitted in a draft license application for review and comment. A joint agency meeting is required if a resource has a substantive disagreement with any environmental study result or conclusion, proposed mitigation, protection measures, or enhancement measures presented in the draft application.

The third stage of consultation begins with the filing of an application for a new license. The application is filed with the FERC and submitted to the resource agencies which were consulted during the first two stages. Public notices are published by the applicant and by the FERC, inviting interventions and requesting comments on the merits of the application.

Preconsultation considerations. An essential element of a successful application is constructive consultations with resource agencies. Constructive consultations are those that satisfy the objectives of each side. For the applicant, the major objective is to convince the resource agencies that the existing project, including any proposed modifications, provides a reasonable balance between power and non-power uses of the resources. The resource agency's objective is to ensure that the project-related environmental impacts are minimized and that public interest is served. The amount of convincing required and what is meant by "minimized impacts" are defined during these consultations.

The tone of the consultations is influenced by several factors. Two major factors are the relationship between the applicant and agency and their attitudes toward each other. If an applicant/agency relationship has been developed, it is the result of past dealings with one another. Each party has a role in developing this relationship and can influence it either positively or negatively by its actions.

The attitudes of the applicant and agency are a complicated matter and can be a direct result of past consultations. However, even if there have been little or no past dealings between the parties, attitudes will be developed from each party's perception of the other. The agency's perception of the applicant is influenced by such things as the applicant's public reputation, past actions, and even past dealings with other agencies. The applicant's attitude toward an agency will be based on agency dealings with other licensees and its own attitude regarding the relicensing effort in general.

Empire had limited dealings with resource agencies prior to the start of the relicensing effort. Empire had recently cooperated with the Missouri Department of Conservation in developing reservoir drawdown criteria. On a local level, Empire has always attempted to be a good citizen and neighbor. Empire maintains a park near the dam, has cooperated with local cities in leasing land for public use, and has maintained its facilities original appearances as much as possible. Overall, Empire felt that its perception by agencies and the public was favorable.

A spirit of cooperation among the parties encourages fruitful consultations. Empire's intention was to begin the relicensing process with the attitude that consultations do not necessarily result in a zero-sum process in which one party's gain is the other's loss.

First stage consultations. The first stage consultations require the applicant to provide appropriate resource agencies with project related information. The project team referred to this information as an "Initial

Consultation Package" (ICP). The distribution of the ICP signals the beginning of consultations. The ICP includes the site specific information regarding general engineering design, streamflow, water quality, local environment, and any proposed studies.

To aid in the development of this information, Empire and Black & Veatch reviewed Empire's past dealings with resource agencies and local authorities, including information shared, agreements entered, and studies performed. This helped target any potentially controversial issues that could arise during the first stage consultations.

In developing the ICP, agencies were asked to provide information on the environmental setting of the project. Instead of simply issuing an ICP for agency review and then holding meetings, the project team assembled a draft ICP which was used as a basis of discussion for individual pre-ICP meetings with selected resource agencies. Not only did these informal meetings serve to identify agency concerns early in the process, they helped put faces to names in a relatively relaxed atmosphere. The results of this effort were used in the development of the final ICP issued to the agencies.

Regulations under Section 4.51, Title 18, Code of Federal Regulations (18CFR), require the applicant to provide detailed environmental information regarding water quality and fish, wildlife, and botanical resources. The regulations also require information on recreational, historical, archeological resources, land management, and aesthetics. It was determined during pre-ICP consultations that information collected during past agency studies would adequately describe the fisheries resource. However, the agencies requested studies to better define recreational resources, investigate the presence of threatened and endangered resources, and investigate the effects of project operations on water quality.

Second stage consultations. Second stage consultations began with the project team developing plans for the requested studies. Plans were developed to perform a water quality investigation, a threatened and endangered species survey, a cultural resources investigation, and a recreational resources study. A Land Management Plan was drafted to address comments concerning Empire's plans for land management.

FERC regulations require agency consultation only after the draft application, which contains results of the studies, is submitted to the agencies. However, it was decided that if the application information was to meet the needs of the agencies as well as application requirements, the agencies needed to be "in the loop" throughout the collection of

information and the development and completion of studies. Draft study plans were sent to relevant agencies for review and comment and subsequently revised to reflect their recommendations. Study objectives were also discussed. Agencies were invited to observe during certain studies and were provided with raw data during the water quality investigation. Since a large portion of the agency responses to the ICP concerned land management and recreation, a predraft joint meeting was held with relevant agencies to discuss the draft Land Management Plan and the recreational resources study plan.

The benefits of close agency involvement throughout the second stage include a feeling of ownership by the agencies in the relicensing process, studies that meet the needs of the agencies in evaluating the draft application, and a positive applicant/agency relationship.

The draft license application was submitted to the agencies for review and comment in February 1991. Agency comments on the draft application confirmed the benefits of Empire's consultation strategy. Most comments requested clarification of or changes to data and information presentation. Although a few "new" issues were introduced, study results and Empire proposals were largely accepted. Most of the comments on the draft application would require minor revisions or additions. The project team felt only a few comments could potentially result in disagreements, including requests for wetlands enhancement, replacement of flashboards with operable gates, and mitigation of low dissolved oxygen levels in Lake Taneycomo.

FERC regulations require that disagreements be discussed in a joint agency meeting before the filing of the license application. However, to get a better understanding of the more substantive comments, pre-joint agency meetings were held with pertinent individual agencies. In these meetings, the project team presented its understanding of all the agencies' comments and possible responses. Also, the agencies' positions on comments from other agencies were solicited. No attempt was made to influence an agency's position on another agency's comments, but this information was useful in developing responses and proposals to be presented for discussion at the joint agency meeting.

The joint agency meeting was held on June 18, 1991. The first order of business was to discuss the agreements and understandings resulting from the pre-joint agency meetings. This paved the way for discussions to concentrate on more substantive issues.

Third stage consultations. On August 26, 1991, Empire filed its application for a new license with the FERC and submitted copies to the

consulted agencies. On October 21, 1991, the FERC accepted the application for filing and requested additional information. The scope of the requested additional information was relatively minor and consisted of submitting a project video, developing plans with relevant agencies to enhance wetland sites, and developing an additional hiking trail within the project boundary. Since Empire had agreed in principle to the wetland and trail issues during the preapplication consultations, developing these plans with the agencies was relatively easy.

In February 1992, representatives of Empire and Black & Veatch traveled to Washington DC to meet the FERC staff members evaluating the application. FERC staff members possessed a good command of the application and exhibited a willingness to discuss specifics of the additional information request. These discussions were helpful in developing responses that met the needs of the FERC staff. While the relicensing process does not require this meeting, it helps to personalize the process and emphasize how seriously the applicant views the application.

The FERC's solicitation of agency comments resulted in a number of recommendations. Although a response by Empire to these recommendations was not required by FERC regulations, Empire felt it best to provide the FERC staff with detailed information regarding its position on each recommendation. While the effects of this additional voluntary action are debatable, this action did provide the groundwork for post license consultations on special license articles.

Issue tracking. The three stage consultation process and the amount of information required to file an application can result in hundreds of agency issues and concerns. Regulations require that agency letters containing comments and recommendations be included with the license application, and that the applicant show that these comments and recommendations have been addressed. Appreciating the importance of keeping track of these issues, a formal method to track an issue's progress was initiated at the beginning of the consultation process. Issue tracking involved keeping a database on all agency issues. The database contained relevant information on the agency, date of letter or memo, and the current status of the issue. Obvious recommendations or concerns, as well as vague references and statements that could be construed as concerns were listed. These issues became the focus of the project team's consultations.

As consultations progressed, the database was continually updated to indicate the status of issues or to add new issues. Reports from the database were used in prompting discussions at project team meetings. An issue was not considered resolved until a letter, meeting, or telephone

memorandum which attested to the resolution was received. Thus, the database prompted the project team to resolve issues at meetings and request verification of solutions by letter if any uncertainties persisted. The issue tracking system proved to be an invaluable tool in tracking agencies' concerns and assured the project team that nothing was being overlooked.

Application Preparation and Submittal

As consultations and studies were conducted, the project team put together the license application. While federal regulations require a detailed and voluminous filing, they are silent regarding a specific format for presenting each exhibit. The project team used the format and organization of the regulations in Sections 4.51 and 16.10 of 18CFR. The regulations for each exhibit were presented verbatim in bold letters, followed by Empire's information responses. This offered a simple method for information presentation, and more importantly, aided the FERC, agencies, and project team in verifying that the required information was presented. This method also broke the application into discreet, stand alone sections, which allowed for internal review of one or several sections at a time.

Section 16.8(f) of 18CFR requires the applicant to include copies of agency letters containing comments, recommendations, and proposed terms and conditions. To aid the FERC and agencies in their review, Empire's responses were pasted alongside the agency letters and reduced to fit on one page. This supplied line-by-line verification that each agency issue had been addressed.

Throughout the preparation of the application, each section was thoroughly reviewed by each member of the project team as it was completed. Application sections regarding known information were drafted early, while most of the Exhibit E sections were drafted as information was gathered and studies were completed. Engineers and attorneys reviewed sections written by biologists and accountants and vice versa. This provided a "layman's" check of the readability of the section. While this review is important, individual sections of the application are not meant to stand alone, but are interrelated in the context of the complete license application. Agency and FERC staff reviewers should be able to read the application and understand all aspects of the project without requiring more information or becoming confused. Therefore, the importance of the in-house review of the complete draft and final license application cannot be overemphasized. This review allows evaluation of each section in the context of the entire application. Also, information presented in early drafts could be revised as a result of studies and agency

consultations. Recognizing the importance of this review, the project team spent many long hours reviewing and discussing the draft and final application. A number of revisions were made to the in-house draft to fill in "holes" and clarify minor inconsistences. The importance of the contextual review of the application may seem obvious to the reader, but schedule or budget constraints can tempt an applicant to skip additional reviews. Such a shortcut may lead to numerous agency questions, comments, and voluminous additional information requests. An hour of preapplication review could prevent many hours spent on additional agency consultations.

The completed application was filed with FERC and submitted to the agencies on August 26, 1991.

Post License Observations

On March 31, 1992, Empire District Electric Company was issued a new 30-year license for the Ozark Beach Project, only seven months after application filing. In conversations with the FERC staff, Empire was complimented on its "clean" and "thorough" application. While Empire had no "sleeping giant" environmental issues to contend with, that was no guarantee that a new license would be issued quickly. Application deficiencies and disagreements on even minor environmental issues can delay the issuance of a license while the applicant is expending time and money to provide additional information and consult with resource agencies.

No single area can be credited as being responsible for the success of the Ozark Beach relicensing project. No magic procedures were implemented in the preparation of the application. The project team aggressively consulted with resource agencies in a spirit of cooperation and the resource agencies exhibited professionalism in their timely responses. No stone was left unturned in the gathering and development of information. What can be credited is a work ethic that stressed meticulous planning, fervent attention to detail, close coordination, careful management of the schedule, intensive consultations, and a close, positive working relationship between Empire and Black & Veatch. This exhaustive effort resulted in not only a new FERC license but relationships with resource agencies that should prove beneficial to all for the next 30 years.

Learning from Relicensing:
Lessons for the Hydro Industry

Marla Barnes[1] and Tom DeWitt[2]

Abstract

To discover lessons learned and trends established through the current relicensing effort, Hydro Review and the Federal Energy Regulatory Commission (FERC) collaborated for this paper to analyze the information resulting from a review of the 1993 relicense applications. This paper highlights significant findings and offers suggestions that can be useful to the industry in future relicensing work, including a proposed new approach to the licensing process.

This paper provides information useful to all segments of the hydroelectric industry: project owners and operators; consultants who offer services and perform studies in support of the licensing process, or who can perform needed studies; and suppliers of equipment and services needed for efficient, safe, and environmentally sound operation of hydroelectric projects.

Introduction

Hydropower, the primary contributor of renewable energy in the U.S., supplies about 13 percent of the nation's electrical capacity, and has sizable potential for growth. Over the next 20 years, the U.S. will need approximately 200,000 MW of new electrical generating capacity.

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[1]Marla Barnes is the Editor of Hydro Review magazine, 410 Archibald Street, Kansas City, MO 64111.
[2]Tom DeWitt is Chief of the Planning and Support Branch in the Office of Hydropower Licensing at the Federal Energy Regulatory Commission, 810 First Street, N.E., Room 1018, Washington, DC 20426.

Hydropower can effectively provide the solution to meeting a meaningful portion of this growing demand.

In addition to helping meet future electricity needs, hydro plants play a critical role in water management. Not only do hydro facilities generate electricity, they often are a key part of projects that provide flood control and water for irrigation/agriculture, navigation, municipal drinking water, recreation, fish and wildlife, and industrial uses. Management of water resources to accommodate these many uses needs to be optimized, and hydro plant owners can play an important part in that management.

As the hydroelectric industry looks toward future opportunities, it needs to take a fresh look at existing projects -- identifying ways to enhance both power production and non-power values.

In 1993, operating licenses for 157 nonfederal hydroelectric projects expire. (These projects provide about 4,000 MW of capacity and are worth about $10 billion at their replacement cost.) Owners of these projects are currently working through the relicensing process to secure new licenses from the Federal Energy Regulatory Commission. Through this process, issues facing the industry are being brought into focus. And hydro plant owners are working with resource agencies and the public to choose operating alternatives that provide the greatest overall public benefit.

The hydroelectric industry can learn from this current relicensing effort. While the large number of projects in the 1993 relicensing class is atypical, relicensing will be a prominent activity for decades to come. In fact, more than 300 hydroelectric projects are candidates for relicensing over the next ten years. Nearly half of these projects have more than one development (a development is defined as a distinct reservoir with an individual power element). Therefore, an awareness of how to best approach the process is needed.

Statistics from the Class of '93

The "Class of '93" refers to the group of hydroelectric projects whose original licenses expire in 1993. As of December 31, 1991, 157 applications had been filed with FERC for new licenses (relicenses). (The Electric Consumers Protection Act stipulates that an application for a new license must be filed with FERC two years before the original license expires.)

Thirty applications (nearly 20 percent of the Class of '93) came from projects in Wisconsin; another 40 applications were from projects evenly divided between Michigan and Maine; and another 17 applications were from projects in New York. FERC staff observed that, for the most part, agencies in states with several projects in the relicensing process were the best organized and prepared for participating in the process.

The number of applicants proposing capacity additions was surprisingly low. Only 24 applications proposed capacity additions, totaling 132 MW.

As of February 1, 1993, FERC has issued new licenses to four projects. FERC staff has sent initial response letters to all other applicants, and accepted 136 of those applications for processing. Most applicants are conducting additional scientific studies and/or developing additional information. Environmental assessments (EAs) for eight projects have been completed, and 15 more are being prepared. One environmental impact statement (EIS) involving seven projects is being prepared.

Of the 157 cases, FERC staff chose to send 47 to Stone & Webster Environmental Services, the lead contractor providing environmental and engineering support services to FERC's Office of Hydropower Licensing. The applications sent to Stone & Webster involve one or more of the following:

-- A potential "major federal action" as defined in the National Environmental Policy Act (NEPA) that will most likely require preparation of an EIS;

-- A cumulative analysis (in which the effects of the project are evaluated in context with several other nearby projects on the same river system); and/or

-- Capacity additions at the project.

For each application, Stone & Webster prepared a "resource information profile" (an internal document summarizing the project) for each application and reviewed the applications for additional information needs (which includes the review of additional scientific study requests). Stone & Webster also has provided FERC with an analysis of the type of NEPA document (an EA or EIS) necessary for each of the 47 applications. FERC staff is reviewing those recommendations, and may make additional assignments to Stone & Webster.

In FERC staff's initial review of the 157 applications, nearly half were found to be deficient (specific items called for under FERC regulations were not provided). Most of the deficiencies either were oversights by the applicant or situations where the applicant clearly did not understand what FERC required. Non-power deficiencies were most prevalent, including 44 applications with recreation deficiencies and 14 with biological deficiencies. As of February 1, 1993, almost all deficiencies have been dealt with by the applicants.

In reviewing the applications, FERC staff and its contractor found that all but nine of the Class of '93 applications -- 94 percent -- needed additional information in order for the application to be evaluated, including information gleaned from additional studies. FERC staff was not surprised that a high percentage required additional information; it had estimated that 85 percent of the applications would need additional information.

Areas of the application that most often lacked adequate information included: recreation, biological aspects, water use and quality, cultural aspects, and cost and financing.

Nearly 900 additional scientific study requests were filed by federal and state agencies, organizations (special interest groups such as American Rivers, Trout Unlimited, and National Wildlife Federation, and individuals such as adjacent property owners), and Indian tribes. (These are requests made after the application was filed with FERC.) FERC staff found that 304 of these studies were actually necessary, affecting 100 projects. (It is interesting to note that approximately 70 percent of all requests filed were associated with only 30 projects.)

Of the 304 necessary scientific study requests, federal agencies requested 40 percent of the studies; stage agencies, 33 percent; organizations, 22 percent; and Indian tribes, 5 percent. (State agencies and organizations submitted the bulk of requests for studies determined to be unnecessary.)

The necessary study requests range from the simple to the complex. They include things such as estimates of potential recreation uses; compliance with the Americans with Disabilities Act; instream flows and reservoir fluctuations; effect of proposed operation on recreation, aesthetics, and public safety; soil conservation at projects proposing modifications; studies of run-of-the-river operation as an alternative to peaking; and fish entrainment. In addition, in some instances decisions were reached without scientific basis, and FERC staff requested the applicant to establish that basis.

Eighty project applicants have filed responses to some of the additional information requests; so far, FERC staff has found that about 20 of the responses are deficient or need further additional information. In addition, at the end of 1992, 74 of the 80 project applicants had requested extensions of time to file the remainder of the requested additional information. As of February 1, 1993, FERC staff has granted 63 of the requests.

In July 1992, FERC staff's goal was for the commission to act on 50 percent of the Class of '93 applications by December 31, 1993. However, time extensions for filing requested additional information are altering this projection. As of February 1, 1993, FERC staff estimates action on only 17 percent of the applications by the end of 1993. The remaining projects would receive annual licenses until new licenses are issued, now estimated to be completed by the end of 1995.

Concerns with the Statistics

Several statistics from the Class of '93 are disconcerting: half the relicensing applications were judged deficient, 94 percent required additional information (including additional studies), and more than 80 percent of the projects probably will not have new licenses by the end of this year. FERC concludes that the current process does not work sufficiently well. Problems identified by FERC include: FERC is a passive participant in the process (staff is not available to answer questions and resolve disputes); disputes among applicants and agencies and/or special interest groups were not resolved before the application was filed; and the public was largely excluded.

In interviews with FERC staff, hydropower developers, and resource agencies, one prominent recommendation that came forth for improving the licensing process was for FERC to be involved before a license application is filed. Consequently, FERC's Office of Hydropower Licensing has proposed a new approach to the license application process that involves FERC staff in pre-filing consultation. At the time of the writing of this paper, FERC was seeking comments on this proposal from hydropower developers, federal resource agencies, state resource agencies, interest groups, and consultants.

A Proposed Solution: CAP

The approach, called the Consolidated Application Process, or CAP, involves FERC staff earlier in the process of preparing a hydro license application. It would reverse FERC's hands-off policy during the pre-filing consultation stage, a position established by licensing rules approved in

1991. The rules require potential applicants to work with resource agencies to prepare a license application, but do not involve FERC staff (with the exception of the director of the Office of Hydropower Licensing in dispute resolution) until the application is filed.

Under CAP, FERC staff would, at a minimum, attend an initial consultation meeting to become familiar with project details, a study plan meeting to determine what studies are appropriate, a mid-course meeting to evaluate the results of the applicant's first year of studies, and a draft application review meeting to discuss the application and draft EA or EIS.

The CAP also greatly increases the number of opportunities the public has to be involved in the process -- under the proposed process, the license applicant would be required to hold four formal public meetings, compared to the one meeting currently required.

Major milestones in the CAP do not differ substantially from those in the current process. What would change is how the milestones are reached and who participates in reaching them.

FERC's goals under the new approach include: 1) Final license applications would be free of deficiencies and requests fo additional studies; and 2) FERC would issue an order within one year from the time an application is filed.

Initial reaction to the Consolidated Application Process has been mixed. Respondents indicated that the strengths of the program include earlier FERC involvement, quicker resolution of issues and quicker processing, and earlier initiation of NEPA. However, respondents expressed concerns about the need for FERC staff to take an authoritative, active, and unified role in the process, and the need for flexibility in the time frames set for accomplishing the various phases of CAP. They also questioned whether FERC could meet the goals set under this new approach. Several respondents pointed out that without a chance in the licensing "mind-set" on the part of the various players, the CAP process likely will be as ineffective as the current process.

FERC plans to continue to refine the process in 1993, incorporating suggestions and reactions from those involved in the licensing process. FERC also is incorporating part of the CAP process in a third-party contracting test case involving a proposed pumped-storage project in Oregon. Possible formal implementation of CAP could come within a year. By attempting to address problems identified in the Class of '93 applications, FERC is proposing a new way of doing business in hydro licensing.

STUDY REQUESTS FOR THE CLASS OF '93 HYDROPOWER PROJECTS

Lee Emery [1]

Abstract

The Federal Energy Regulatory Commission (Commission) received 889 requests for scientific studies from various entities for 157 license applications filed by licensees seeking to relicense their hydropower projects. Licenses for all 157 projects expire by December 31, 1993. Over half of the projects are located in the Great Lakes and New England areas.

Most of the study requests wanted information on biology, recreation, and aesthetics. The state resource agencies had the most requests (346), followed by Federal Agencies (239), other organizations (265), and Indian Tribes (39). About 66 percent (585) of the study requests didn't meet the Commission's regulatory requirements. However, much of the same type of information was later requested by us in letters asking for additional information.

This paper includes a brief discussion of reactions to our handling of the study requests and ways to improve our handling of the next group of relicenses arriving in the late 1990's.

Characterization of Projects and Commenters

Near the end of last year, licensees filed applications to renew their licenses for 157 hydropower projects scattered across the country (figure 1). Over half of the projects are located in the Great Lakes and New England areas. Most of these hydropower projects

[1] Chief, Aquatic Resources Section, West Branch, Division of Project Review, Federal Energy Regulatory Commission, 810 First Street, Washington, D.C. 20426

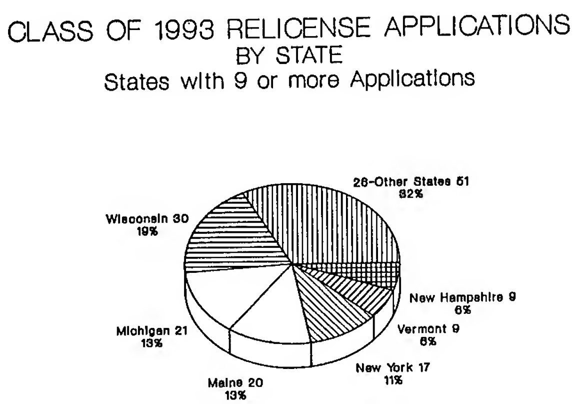


Figure 1. Class of '93 relicense applications filed by various states (states having 9 or more applications).

have been operating for nearly 50 years and all have licenses that expire on or before December 31, 1993 (The Class of '93). The law requires these licensees file an application for a new license with the Commission two years before their present license expires.

Licensees have to follow a detailed and lengthy process before filing their applications with the Commission. They have to do such things as consult with state and federal resource agencies, Indian Tribes, and hold public meetings, conduct studies, and make information about their projects available to the public.

This paper discusses one aspect of this filing process: our (Commission staff) treatment of study requests made by resource agencies, Indian Tribes, and other groups on those license applications filed before or during December 1991. Before discussing our treatment of the study requests, however, I will briefly describe how study requests can be provided by various entities.

Submission of Study Requests by Resource Agencies

There are many formal opportunities for resource agencies (including Indian Tribes) to ask for studies on license applications before they are filed. Of course there's always an opportunity for informal discussions between applicants and resource agencies too. The picture I'm trying to paint is that there's more than one opportunity for resource agencies to take a bite out of

the apple, so to speak.

Basically, there's a three-stage consultation process that an applicant must follow before filing a license application with the Commission. The resource agencies and Tribes have at least three different opportunities to provide comments during the three-stage process.

The first stage can be started anytime within 5 years before a license expires. During the first stage of consultation, applicants must provide the agencies with an information package about the project. The information package contains such things as project boundary maps, a general engineering design of the project, an identification of affected resources, stream flow data, and any proposed studies.

After providing this information, the applicant must hold a joint meeting and site visit within 30 to 60 days after transmitting the information package to the agencies and Indian Tribes. Members of the public are invited to attend the joint meeting and can participate fully in the meeting by expressing their views on how resource issues should be addressed in any application for a new license that is filed by the applicant.

The resource agencies come to the meeting prepared to (1) discuss current and prospective resource needs, (2) their management objectives for resources in the project area, and (3) what information and studies they believe are needed. Resource agencies and Tribes must supply their comments (including study requests) on the proposed project no later than 60 days after the joint meeting. The first stage ends after the agencies and Tribes supply their comments on the proposed project or no later than 60 days after the joint meeting.

The second stage begins with the licensee conducting studies and gathering information requested by the resource agencies during the first stage of consultation. The licensee assembles the information and the results of studies and prepares a draft application for agency review. The second stage ends when the resource agencies provide comments to the licensee after a joint meeting is held (if there is substantial disagreement) or within 90 days after the resource agencies receive a copy of the draft license application.

The third stage begins when the licensee files an application with the Commission and the consulted agencies. It is at this point that the we begin processing the license application.

Stage 3 was completed on December 31, 1991, when 157 out of 167 total license applications were filed with the Commission. (Ten projects were not timely filed for various reasons). It was at this time, after we had publicly noticed the filings in the Federal Register and requested comments, that we received the 889 additional study requests I'll be discussing in this paper.

Submission of Study Requests by the Public

Informally, the public can talk to the licensee at any time. As mentioned earlier, the public can also participate in the public meeting held during the first stage of consultation.

The public has their first chance to formally submit study requests after a license application is filed with the Commission and we issue a public notice asking for requests for additional studies. Of course at any time the public can also ask their resource agency representative to address environmental issues of mutual concern and to bring those concerns to the attention of the licensees.

Discussion of 889 Study Requests

Various entities for the 157 hydropower projects comprising the Class of '93 filed 889 additional scientific study requests (figure 2).

CLASS OF 1993 RELICENSE APPLICATIONS
Sources of 889 Additional Study Requests

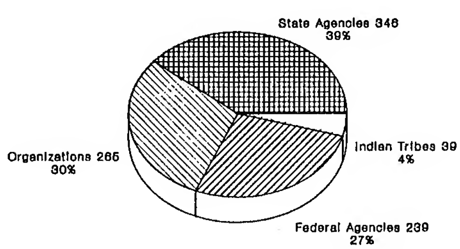


Figure 2. Sources and numbers of study requests filed by various organizations for license applications for the Class of '93.

We were surprised at the large number of study requests; we'd seen only a few disputes arise between

resource agencies and applicants over the need for studies during the processing of applications before the December 31st filing deadline. We hadn't detected this flood of study requests during the outreach meetings we held over the years. In these outreach meetings, we told licensees how to prepare an application that would be acceptable for filing. We also told the resource agencies and the public what was expected of them and how they could participate in the process. Perhaps the large number of study requests shows how well we did our job in educating the participants, or maybe it shows how changes in regulations (Order 533 was issued in June 1991) allowed the agencies and the public to file study requests. Actually, the receipt of the large number of study requests does not mean that all applications missed the mark, because roughly 70 percent of all 889 study requests were associated with 30 projects.

Identification of Study Request by Entity

The study requests were made by many different entities. The state resource agencies requested the most, about 39 percent. The breakdown by request for each entity was:

- Federal agencies, 239 (27 percent);
- State agencies, 346 (39 percent);
- Other organizations, 265 (30 percent); and
- Indian Tribes, 39 (4 percent).

We agree with the requesters on 304 of the 889 requests for additional studies; we've asked the licensees to conduct these studies (figure 3). The breakdown by entity for the 304 study requests we agreed with and required the licensees to study were:

- Federal agencies, 122 (40 percent);
- State agencies, 99 (33 percent);
- Other organizations, 67 (22 percent); and
- Indian Tribes, 16 (5 percent).

We felt that 585 study requests, or about 66 percent of the total requests, were unnecessary (figure 3). I'll discuss why in a few moments. In general, the breakdown of the study requests that we didn't accept shows that about 50 percent of federal and Tribal requests were accepted and about 25 to 29 percent of the state resource agencies and other entities' requests were accepted. The breakdown was as follows:

- Federal agencies, 239
    - -necessary, 122 (51 percent)
    - -unnecessary, 117 (49 percent)

- **State agencies, 346**
  - -necessary, 99 (29 percent)
  - -unnecessary, 247 (71 percent)

- **Other organizations, 265**
  - -necessary, 67 (25 percent)
  - -unnecessary, 198 (75 percent)

- **Indian Tribes, 39**
  - -necessary, 16 (41 percent)
  - -unnecessary, 23 (59 percent)

CLASS OF 1993 RELICENSE APPLICATIONS
889 Additional Study Requests And
Number of Requests Found Necessary

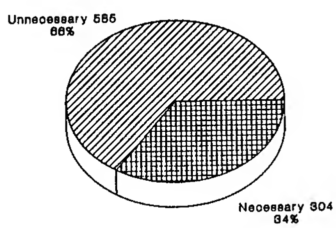


Figure 3. Study requests filed by various organizations, with percent of total requests, for license applications for the Class of '93.

Reasons for Not Accepting Study Requests

Regulatory Reasons

We reviewed each of the study requests to determine acceptability based on the standards set forth in the Commission's regulations. Specifically, we determined if the study requests met the requirements of Section 4.32(b)(7) which required each study request to include a number of things justifying the need for such studies.

Many of the requests didn't meet the standards required by the regulations (figure 3) and we sent letters to the requesters telling them that their requests were not necessary to complete our analysis of the project. However, it's not quite as black and white as it appears, because we also considered other factors in our review of the study requests.

The driving force behind our review of the study requests was that we needed to determine which studies would provide information necessary for us to form an adequate, factual basis for completing an analysis of a project based on its merits. Ultimately, the types of studies required from the licensees would provide information that would help us to prepare documents (e.g., environmental assessments and environmental impact statements) that met the requirements of the National Environmental Policy Act (NEPA).

It should be noted that many of the study requests that were found to be "unnecessary" as presented by requesters because they didn't meet the regulations, had already been determined to be needed by us in an independent review of the license applications. We had thoroughly reviewed the license applications before receiving the study requests and had a good understanding of what information we would require from the licensees. We sent additional information request (AIR) letters to the licensees giving them anywhere from a few months to about a year to provide the additional information.

<u>Other Reasons</u>

Other study requests were found to be unnecessary because the information to be collected from the study was either present in the application or the information requested would be answered in some other related study request (either made by another requesting entity or by us in our AIR).

I should mention that many of these license applications were big, voluminous documents, some containing over a cubic foot of material for an individual project. So when reviewing these applications it was easy to overlook some detail that might answer a question that was asked in a study request. That answer may have been located in another part of the license application, for example, in an appendix of a volume not related to the topic of concern.

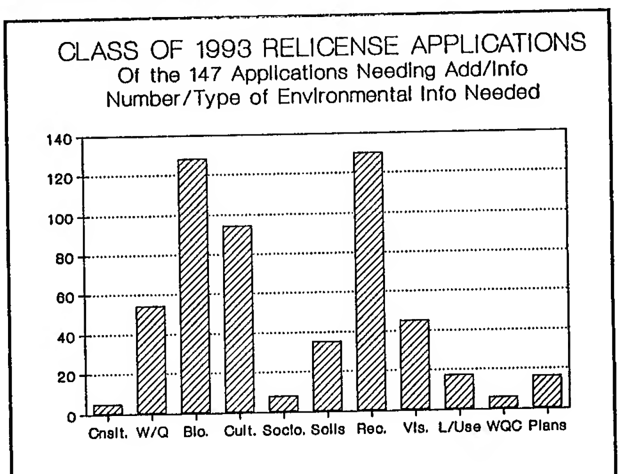
<u>Types of Additional Study Requests</u>

My review of the 889 study requests showed that biological information, recreation, and aesthetics were generally the three subject areas receiving the most study requests by all entities (Table 1). The listing of the top 10 most requested studies is based on a random sample of 514 requests (58 percent of the total number of requests).

Figure 4 shows how closely the requesters study requests were to our AIR's and how everyone had a need

Table 1. Top 10 most frequently requested study requests by all entities (excluding the Commission's staff) for the Class of '93. (Actual numbers of requests are in parentheses).

- MINIMUM FLOWS (74)
- IMPINGEMENT & ENTRAINMENT (66)
- WATER QUALITY (60)
- RECREATIONAL USE (56)
- IFIM STUDIES (49)
- WHITEWATER RECREATION (44)
- FISH ENHANCEMENT (44)
- WATER LEVEL FLUCTUATIONS (42)
- VISUAL QUALITY/AESTHETICS (41)
- WILDLIFE HABITAT (38)



Key

Cnslt.=Consultation with agencies
W/Q=Water Quality
Bio.=Biological Resources
Cult.=Cultural Resources
Socio.=Socioeconomics
Soils=Geology and Soils
Rec.=Recreational Resources
Vis.=Visual Resources
L/Use=Land Use
WQC=Water Quality Certificate
Plans=Comprehensive Plans

Figure 4. Types and numbers of environmental requests sent in additional information requests by the Commission's staff to 147 licensees filing license applications for the Class of '93.

for information in the same subject areas for many projects. Out of the 147 applications needing additional information, we found the most frequent need for information was in three areas: recreation, followed closely by biological information, and cultural resources (figure 4). The difference between our request for additional information (figure 4) and the requests for studies made by all other entities (Table 1.) occurred in three areas: cultural resources, aesthetics, and recreation. There was little difference between our requests for recreational and biological information and the requests made by others (water related concerns were first through third and recreation was fourth)(Table 1).

The large number of requests for recreational resources (figure 4) by us is somewhat influenced by our need for information showing how the licensees would meet the 1990 Americans With Disabilities Act; almost every licensee had failed to address this concern in their application. In addition, most licensees failed to provide information on how much their recreational enhancement measures would cost.

Our large number of requests for additional information on cultural resources is probably not surprising if you think about it. Many of these projects have facilities that are over 50 years old and therefore are eligible for listing on the National Register of Historic Places. The licensees did a pretty good job of describing what these facilities were, but failed to address how these historic structures and the archeological resources around the reservoirs would be protected or how any potential project impacts on these resources would be mitigated.

Reactions to Our Actions on the Study Requests

Resource Agencies

As you might expect, the reactions to our actions on the study requests were mixed. Generally, we don't get direct feedback on these types of matters. To find out what others thought, I contacted a couple of state resource agencies and asked them their candid opinion on how they thought we handled their requests for studies. The comments went something like this: "FERC was arbitrary and capricious"; "The process was confusing"; and "FERC's response letters back to us (the resource agencies) were terse and not very helpful".

I think if the resource agencies were to take a closer look at the outcome of all their study requests they'd be pleased--because as I mentioned earlier--we required the licensees to conduct many of the requested

studies in our AIR's.

Public Interest Group

One public interest group, the American Whitewater Affiliation (AWA), reported in their JULY/AUG 1992 issue of American Whitewater Magazine that they felt they had been heard and were pleasantly surprised by FERC's "serious interest" in evaluating recreational needs in the relicensing process.

Licensees

The licensees reaction to the study requests were about as colorful as the resource agencies comments. They said such things as: "The industry was bowled over"; "It seems more than coincidental that study requests rejected by FERC would reappear later as an 'independently' developed item that was needed"; "FERC the umpire, said 'you're out'--but not really"; "We did not expect to be lockstep with the resource agencies"; "The study requests have a aura of academia"; and "We had enough information in the application".

You can see from these comments that the industry was not pleased with our handling of the study requests. As I said earlier, and I want to mention again because of its significance, the reason behind our request for the studies was to obtain information to form an adequate and factual basis to complete an independent analysis of the project based on its merits.

Lessons Learned[2]

We would probably better serve those involved in the licensing process by participating in some of the prefiling consultation meetings in the future. Our participation early in the process might avoid another barrage of study requests after the applications are filed with us. (There are about 80 relicense applications expected to be filed in the late 1990's). Earlier participation by us might also ensure that the resource agencies are making meaningful study requests during the first stage of consultation and that the licensees have really heard what was said. Our participation in the first stage consultation meetings

2 Any opinions or views I may offer here or elsewhere in this paper are my own and not those of the Office, the Commission, individual Commissioners, or other members of the Commission staff.

would also: 1) give us an opportunity to hear the concerns of the public early in the licensing process and 2) help ensure that information we need to analyze a project and to prepare NEPA documents is given to us in the license application.

If we use the proposed Consolidated Application Process (CAP) presently under consideration, there would be an improvement in the prefiling process for license applications.

Acknowledgements

I thank Tom Dean and Ray Feller for their technical review of this paper and John Mitchell for his editorial review.

The Demise of "Equal Consideration"; Otherwise Known as ESA Section 7 Consultation

Gary D. Bachman
Cheryl M. Feik[1/]

I. Abstract

With the Electric Consumers Protection Act ("ECPA") of 1986, the Federal Energy Regulatory Commission ("FERC") is required to give equal consideration to developmental and nondevelopmental values in determining whether to issue new hydroelectric licenses. ECPA requires that FERC consider licensing conditions proposed by fish and wildlife agencies. If it declines to make them a condition of the license, FERC must formally explain its refusal to do so. Its ability to balance competing interests under ECPA, however, is seriously undermined to the extent that licensing decisions are being driven by the inflexible requirements of Section 7 of the Endangered Species Act. Reconciling the policies of ECPA and the Federal Power Act with the demands of the Endangered Species Act generates a variety of issues for relicensing applicants whose projects affect in any way endangered or threatened species. This discussion considers the appropriate substantive and procedural role of the Endangered Species Act in the relicensing process and advocates that Section 7 consultation be effectuated as early as possible in relicensing proceedings so as not to disrupt FERC's efforts to balance competing interests.

II. Standards for Relicensing under the Federal Power Act

Prior to the enactment of the Electric Consumers Protection Act ("ECPA") in 1986, the Federal Power Act authorized the Federal Energy

1/ Mr. Bachman is a Member, and Ms. Feik is an Associate, of the law firm of Van Ness, Feldman & Curtis, A Professional Corporation, 1050 Thomas Jefferson Street, N.W., Washington, D.C. 20007.

Regulatory Commission ("FERC" or "Commission") to issue a license whenever a hydroelectric project was "in the judgment of the Commission, desirable and justified in the public interest for the purpose of improving or developing a waterway or waterways for the use or benefit of interstate or foreign commerce."[1] The passage of ECPA did not alter this standard, but was instead aimed at focusing FERC on its obligation to balance developmental and nondevelopmental interests. The added language provided that:

> [i]n deciding whether to issue any license . . ., the Commission, in addition to the power and development purposes for which licenses are issued, shall give equal consideration to the purposes of energy conservation, the protection, mitigation of damage to, and enhancement of, fish and wildlife (including related spawning grounds and habitat), the protection of recreational opportunities, and the preservation of other aspects of environmental quality.[2]

ECPA also added language requiring FERC to include conditions for the protection, mitigation, and enhancement of fish and wildlife when licensing projects in order to "adequately and equitably" protect, mitigate, and enhance fish and wildlife affected by the projects. These conditions were to be based on recommendations received from fish and wildlife agencies. If, after analyzing the recommendations, FERC believed that they were inconsistent with the purposes of the Federal Power Act or other applicable law, it was to work with the agencies to resolve the inconsistencies. FERC was given the latitude, however, to reject agency recommendations by formally explaining why the recommendations were inconsistent with the purposes of the Federal Power Act. In doing so, FERC was required to explain why the conditions selected did indeed meet the statutory requirements of protecting, mitigating, and enhancing fish and wildlife.

As amended, there is no doubt that the statute requires the Commission to "give significant attention to, and demonstrate a high level of concern for all environmental aspects of hydropower development."[3] As a practical matter, however, ECPA was not intended to elevate concerns for fish and wildlife over those for power needs and economics.

Unlike other sections of the Federal Power Act that specifically authorize other federal agencies to impose conditions on FERC licenses,[4] Congress did not authorize fish and wildlife agencies to directly condition a FERC license. The significance of this cannot be overlooked: in 1990, the D.C. Circuit properly held that the Federal Power Act cannot now be read to force upon FERC the burden of strict acceptance of each and every recommendation made by fish and wildlife agencies. The court recognized that:

> [w]hile the Commission must address each recommendation, the discretion ultimately vests in the Commission as to how to incorporate each recommendation. If we read the statute any other way, the Commission would be held hostage to every agency recommendation, and the Commission's role of reconciling all competing interests would be compromised.[5]

The passage of ECPA did not change the standard governing the licensing projects by the Commission when reconciling competing interests. Relicensing proceedings at FERC have long been driven by the "public interest" standard. As established by the Supreme Court over twenty years ago, "public interest" includes the interest in "preserving reaches of wild rivers and wilderness areas, the preservation of anadromous fish for commercial and recreational purposes, and the protection of wildlife."[6] ECPA merely set forth a procedure for soliciting and for adopting or rejecting recommendations from resource agencies regarding environmental mitigation and enhancement measures.

ECPA does not prescribe standards for giving "equal consideration" and the courts have left to FERC the best manner in which to incorporate agency recommendations. In addressing arguments that FERC failed to give due consideration to recommendations made by the National Wildlife Federation pursuant to the Federal Power Act, the D.C. Circuit found that "[a]lthough the Commission is required to give equal consideration to environmental values and the need for development, it is not necessarily required to give these sets of competing values equal weight in every situation."[7] Similarly, when considering claims that FERC did not give equal consideration to the relevant factors in its relicensing decision, the Second Circuit found that where there is evidence that FERC was aware of the issues presented, and that it considered them in the context of the decision as a whole, FERC has fulfilled its obligations under the Federal Power Act.[8]

The latitude to reject agency recommendations after careful consideration is crucial if FERC is to continue to reconcile a variety of competing interests. Since ECPA, FERC has gone to extraordinary lengths to attempt to balance competing developmental and nondevelopmental interests by striking an equilibrium between the licensee's interests and the recommendations of the fish and wildlife agencies in its relicensing decisions. In doing so, FERC has both rejected recommended minimum flow levels which would force an employer out of business,[9] and imposed minimum flow levels even though they would impact adversely on property owners.[10] Its ability to continue its intended role of balancing all aspects of the "public interest" under the Federal Power Act is being undercut, however, where the hydroelectric project affects any listed or endangered species protected by the Endangered Species Act.

III. The Effect of Section 7 of the Endangered Species Act on Relicensing

Section 7 of the Endangered Species Act,[11] which is intended to protect endangered or threatened species from the adverse effects of federal agency action, imposes both substantive and procedural obligations on "any department, agency, or instrumentality of the United States."[12] As an independent federal instrumentality, FERC must ensure that the projects it licenses are not "likely to jeopardize the continued existence of any endangered species or threatened species or result in the destruction or adverse modification of critical habitat of such species."[13] As required by the Act, this analysis and determination must be carried out in consultation with the United States Fish & Wildlife Service ("FWS") and the National Marine Fisheries Service ("NMFS").

The regulations governing the consultation process, which were promulgated by FWS/NMFS in 1986, are expressly intended to "serve as an effective tool for the early resolution of potential conflicts involving listed species."[14] The regulations do not mandate the timing of review but allow that such review will be triggered at FERC's sole discretion. Despite this seeming flexibility, FERC and FWS/NMFS have been unable to coordinate their environmental review procedures in a way that allows FERC to carry out its role as mediator in reconciling competing interests under ECPA while at the same time fulfilling its obligation under the Endangered Species Act. The agencies have simply failed to work together for early resolution of potential conflicts involving listed species.

A. Primary responsibility for ensuring compliance with the Endangered Species Act rests with FERC.

FERC has the ultimate duty to ensure that its actions are not likely to jeopardize listed species or adversely modify critical habitat under the Endangered Species Act. FWS/NMFS, on the other hand, perform an advisory function by consulting with FERC to identify and help resolve conflicts between listed species and their critical habitat and proposed licensing decisions. To assist in conforming proposed licensing actions to the requirements of the Endangered Species Act, FWS/NMFS are required to issue a "biological opinion."

Nonetheless, FERC makes the final decision on whether consultation is required and how to proceed once a biological opinion is issued. Unlike ECPA, which allows FERC to reject agency recommendations by formally explaining why the recommendations are inconsistent with the Federal Power Act, FERC's ability to reject recommendations made by FWS/NMFS under the Endangered Species Act is governed by a different standard. Once it is shown that listed species are present in the proposed project area, if FERC declines to follow the recommendations of FWS/NMFS, it must show that it is taking "alternative,

reasonably adequate steps to ensure the continued existence" of listed species.[15] Both FERC and FWS/NMFS are required to use the "best scientific and commercial data available,"[16] which requires FERC's ultimate decision to be based upon credible scientific evidence. For the license applicant, however, it is important to remember that FERC is not the only party who bears the risk of an erroneous decision or a mismanaged licensing proceeding: the licensee must also live with the consequences of any subsequent litigation if the advice of FWS/NMFS is not heeded.

B. FWS/NMFS's biological opinion has a significant impact on the relicensing process.

At the earliest stages of review under the Endangered Species Act, FWS/NMFS play a relatively minor role. Although the agencies may consult or confer with the applicant and FERC initially in order to identify whether any listed species or critical habitat is within the action area, it is generally incumbent upon the applicant to undertake the required studies and upon FERC to prepare a biological assessment after an application for a new license has been filed. The biological assessment is prepared in order to facilitate the assessment of the impact of the proposed project on listed species and critical habitat.

Based upon its findings in the biological assessment, FERC will decide whether to initiate formal consultation with FWS/NMFS. Formal consultation is required if FERC determines that its action adversely affects any listed species or its critical habitat. Any possible effect, whether beneficial, benign, adverse, or of an undetermined character, usually triggers the formal consultation requirement. Because "adversely" is defined so broadly and because the list of endangered and threatened species is so vast, it is a rare instance when a hydroelectric project will not potentially affect endangered or threatened species, and, therefore, not be subject to the scrutiny that comes with formal consultation.

Formal consultation between FERC and FWS/NMFS is critical because it may result in a jeopardy opinion. A jeopardy opinion is issued when a project is found likely to jeopardize the continued existence of listed species or result in the destruction or adverse modification of critical habitat. Such a determination will greatly affect the conditions that will ultimately be attached to the issuance of a license by FERC.

The biological opinion issued by FWS/NMFS will include: (1) a summary of the information on which the opinion is based; (2) a detailed discussion of the effects of the action on listed species or critical habitat; and (3) the FWS/NMFS's opinion as to whether the action is likely to jeopardize the continued existence of a listed species or result in the destruction or adverse

modification of critical habitat. FWS/NMFS will then reach one of three conclusions in the opinion: (1) the action is not likely to jeopardize the continued existence of listed species or result in the destruction or adverse modification of critical habitat (a "no jeopardy" opinion); (2) the action is likely to jeopardize the continued existence of listed species or result in the destruction or adverse modification of critical habitat (a "jeopardy" opinion); or (3) the action is likely to jeopardize listed species or result in adverse modification of critical habitat but there are "reasonable and prudent alternatives" to the proposed action which would not result in the likelihood of jeopardy or adverse modification and which can be taken by FERC or the license applicant (a conditioned "no jeopardy" opinion).

In fact, a jeopardy opinion is rarely, if ever, issued by FWS/NMFS in the context of the Section 7 consultation. Formal consultation has evolved into a process whereby FWS/NMFS make it known to FERC that it must incorporate certain additional licensing conditions in order to avoid a "jeopardy" opinion. Once FERC agrees to incorporate the licensing conditions recommended by FWS/NMFS, a conditioned "no jeopardy" opinion is issued. In this way FWS/NMFS avoid issuing a "jeopardy" opinion as well as ever having to justify its recommendations as "reasonable and prudent." Unfortunately, the license applicant does not play a significant role during these negotiations between the agencies.[17]

FWS/NMFS will conclude that a proposed project license will jeopardize the continued existence of listed species if the project is expected, either directly or indirectly, to reduce "appreciably" the "likelihood" of both the survival and recovery of a listed species in the wild by reducing its reproduction, numbers, or distribution.[18] In making its determination, FWS/NMFS will consider whether a sufficient number of individuals or populations, or both, will remain together with sufficient habitat to ensure that the species will "keep its integrity in the face of genetic recombination and known environmental fluctuations."[19] The opinion will find that a project is likely to result in the destruction or adverse modification of designated or proposed critical habitat if it finds that there is an alteration that appreciably diminishes the value of critical habitat for both the survival and recovery of a listed species.

Following the issuance of the biological opinion, FERC must determine whether and in what manner to proceed. Only if the proposed project receives a "no jeopardy" opinion, or if FERC adopts any reasonable alternative provided in a "jeopardy" opinion, may the licensing proceed in compliance with section 7.

C. *Failure to coordinate environmental review by FWS/NMFS and FERC disrupts the balancing process under ECPA.*

The central problem for the license applicant arises from FWS/NMFS's failure to become substantively involved in section 7 analysis until after FERC issues its Draft Environmental Impact Statement ("Draft EIS"),[20] the environmental review required by the National Environmental Policy Act ("NEPA").[21] As prepared by FERC, the Draft EIS is intended to be the biological assessment and typically identifies the "preferred alternative" (which takes the form of the proposed operating criteria and environmental enhancement and mitigation measures FERC intends to include as license conditions). At this point in time, FERC has already completed, and included in the Draft EIS, its "equal consideration" analysis of all relevant agency recommendations, with the exception of those pertaining to section 7 and has presumably struck its proposed "public interest" balance of developmental and nondevelopmental values. FERC is then faced with two undesirable alternatives when it receives the biological opinion prepared by FWS/NMFS: it either can impose the recommendations received as additional license conditions or it can revisit its "equal consideration" analysis and alter the balance among competing interests already set forth in the Draft EIS. The first alternative is potentially unfair to the license applicant who may see the project's generation capability decline and at the same time its associated costs significantly increase. The second alternative -- which involves altering the initial balance between the parties -- is subject to great political opposition. Once FERC has demonstrated an inclination for one set of license conditions over another, it will undoubtedly face stiff opposition if it tries to change them by eliminating certain environmental obligations to account for the costs of the section 7 consultation.

As the environmental review process is currently structured, FERC's "equal consideration" analysis has already been completed *before* FWS/NMFS even begins their analysis and recommendation process. To better carry out the mandate of ECPA, consultation should be entered into early in the planning stage of a proposal. FWS/NMFS should be required to deliver their biological opinion within FERC's time frame so that the biological opinion can be included in FERC's "equal consideration" analysis. Alternatively, FERC should refrain from setting forth in its Draft EIS the "preferred alternative" it proposes to incorporate in the issuance of a license. Rather than naming a discrete "preferred alternative" it proposes to incorporate as part of its decision of record, FERC should enumerate and discuss all of the potential alternatives. In this way, all interested parties could comment on the proposed alternatives, and, at the same time, allow FERC to retain some flexibility to adjust its proposed license conditions to achieve the required balance of interests once its consultation with FWS/NMFS is completed.

D. *A rebalancing will be required in every licensing decision so long as FERC and FWS/NMFS insist on using different baselines*

The problems associated with the breakdown in the coordination of environmental review by FERC and FWS/NMFS are further exacerbated, from the viewpoint of the license applicant, due to the battle between FERC and the fish and wildlife agencies over the definition of the environmental baseline. This determination is critical in determining the "effects of the action." The baseline sets the framework for subsequent analysis required by the consultation regulations of whether a proposed action may affect listed species or critical habitat, whether it may adversely affect listed species or result in destruction or adverse modification of critical habitat.

FERC has decided that the baseline for relicensing purposes should be governed by the project and its surrounding environment as it presently exists. FWS/NMFS has indicated that it may identify the baseline as the pre-project environment. The regulations define environmental baseline to include "past and present impacts of all Federal, State, or private actions and other human activities in the action area."[22] The impacts of two other groups of actions will also be considered part of the environmental baseline by FWS/NMFS: impacts from all proposed projects in the action area that have already undergone consultation; and impacts from state or private actions which are occurring contemporaneously with the consultation process.[23] However, using the pre-project environment as the standard -- which entails looking back more than fifty years -- will undoubtedly significantly increase the perceived effects of the project that a license applicant will have to address in the Section 7 consultation.

IV. Conclusion

Section 7 of the Endangered Species Act is a powerful tool for those interests who want to alter the structure of hydroelectric projects when they come before FERC for relicensing. Applicants can take no comfort in the absence of FWS/NMFS in the early stages of project review. Applicants for new licenses must take an aggressive role in order to engage FWS/NMFS on endangered species issues as early in the relicensing process as possible. Once the "preferred alternative" is set forth by the Commission, it is politically difficult to go back and change environmental mitigation and enhancement conditions included in FERC's initial balance: the tendency and the temptation is then to make Section 7 consultation additive without regard to the consequences to the public interest balance FERC initially identified.

ENDNOTES

1. 16 U.S.C. § 797(e) (emphasis added).

2. Id.

3. See H.R. REP. NO. 507, 99th Cong., 2d Sess. 22, reprinted in 1986 U.S. CODE CONG. & ADMIN. NEWS 2508-9.

4. See 16 U.S.C.§ 797(e) (authorizes federal land managers to impose conditions on a FERC license for dams within reservation lands); 16 U.S.C. § 823a(c) (authorizes fish and wildlife agencies to impose conditions for conduit facilities, and facilities that use 'natural water features' instead of dam structures); 16 U.S.C. § 811 (allows either NMFS or the FWS to prescribe fishways at any FERC dam).

5. National Wildlife Federation v. FERC, 912 F.2d 1471, 1480 (D.C. Cir. 1990) (emphasis added).

6. Udall v. Federal Power Commission, 387 U.S. 428, 450 (1967).

7. 912 F.2d at 1481.

8. Friends of the Ompompanoosuc v. FERC, 968 F.2d 1549 (2d Cir. 1992).

9. Elkem Metals Co., 41 FERC ¶ 62,289 (1987).

10. Brazos River Authority, 48 FERC ¶ 62,190 (1989).

11. 16 U.S.C. § 1536.

12. See 16 U.S.C. § 1532(7).

13. 16 U.S.C. § 1536(a)(2).

14. See 51 Fed. Reg. 19,926 (June 3, 1986).

15. Tribal Village of Akutan v. Hodel, 859 F.2d 651, 660 (9th Cir. 1988).

16. 16 U.S.C. § 1536 (a)(2).

17. While FWS/NMFS's regulations imply that the license applicant is a full participant in formal consultation, FERC has been using its ex parte rules to limit the applicant's role.

18. 50 C.F.R. § 402.02.

19. 51 Fed. Reg. at 19,934.

20. FERC prepares an Environmental Assessment in lieu of an EIS where it finds no significant environmental impact. For purposes of this discussion, it is assumed that FERC prepares an EIS.

21. 42 U.S.C. § 4321 et seq.

22. 50 C.F.R. § 402.02.

23. Id.

THE BATTLE OF APPOMATTOX:
FEDERAL/STATE CONFLICTS IN RETROFITTING
STATE AND MUNICIPAL WATER IMPOUNDMENTS

M. Curtis Whittaker[1]
and
Mark J. Sundquist[2]

In late October, 1992, construction began on a modest three megawatt hydroelectric project on the Appomattox River near Petersburg, Virginia. This project will develop the hydroelectric potential of the Brasfield Dam, a 55 foot-high, 1,250 foot long concrete gravity dam that impounds the Appomattox River to form Lake Chesdin. The Brasfield Dam is owned and operated by the Appomattox River Water Authority (the "Authority"), a state-chartered public entity vested with the mission of providing a reliable water supply to five separate cities and counties. The Brasfield Dam is a major component of the Authority's water supply system, which includes the dam, reservoir, and water intake and treatment facilities.

The newly constructed "Brasfield Dam Hydroelectric Project" (the "Project") represents a "win-win" situation for energy and environmental interests. Retrofitting the existing dam with a run-of-river penstock and powerhouse creates virtually no new environmental impacts. The new project captures a renewable source of energy that was previously wasted as dam overflow. Revenues from Project operations will in part be used to fund a fish passage system on the dam, overcoming the single largest fish passage barrier on the Appomattox River. The Project typifies the potential for advancing both energy and environmental interests through careful hydroelectric retrofits of existing, publicly or cooperatively owned impoundment and irrigation structures.

Despite the salutary "win-win" benefits of this Project, the Brasfield Dam Hydroelectric Project is being constructed only because it overcame a regulatory

system that actively impedes, rather than encourages, this type of development. This paper describes that regulatory failure, and how it was overcome in this instance. This paper concludes with a prescription for needed changes in the way the Federal Energy Regulatory Commission ("FERC") approaches hydroelectric development at non-federal, publicly owned water supply and irrigation structures.

I. The Problem.

In the early 1980's, a small private hydropower developer filed for a preliminary permit on the Brasfield Dam. This filing made the Authority aware of the dam's vulnerability to third party licensing under Part I of the Federal Power Act ("FPA").[3] In order to maintain control over its dam and water supply system, the Authority eventually filed a competing license application. Given preference due it as a municipal entity,[4] the Authority was granted a license to retrofit the Brasfield Dam with a hydroelectric system in 1988.[5]

While the Authority would later ask FERC to alter several aspects of the license as originally issued, two aspects of the license's terms worked to create the regulatory impasse that nearly derailed the Project. First, the Project "boundary" set forth in the license, delineating the extent of FERC's regulatory jurisdiction with respect to this Project, included not only the new penstock and powerhouse, but also the entire dam, reservoir, water pumps and intake lines. This is common FERC practice today.[6]

Second, FERC included in the license standard "reopener" clauses that made clear that by virtue of the hydroelectric retrofit of the Brasfield Dam, FERC would be the final authority over almost any aspect of the operation, maintenance, or use of water anywhere within the Project boundary - which included the entire dam and reservoir. Typical of these reopener clauses is Article 12 of FERC form L-11, which provides as follows:

> The operations of the Licensee, so far as they affect the use, storage and discharge from storage of waters affected by the license, shall at all times be controlled by such reasonable rules and regulations as the Commission may prescribe for the protection of life, health, and property, and in the interest of the fullest practicable

> conservation and utilization of such waters for power purposes and for other beneficial public uses, including recreational purposes, and the Licensee shall release water from the project reservoir at such rate in cubic feet per second, or such volume in acre-feet per specified period of time, as the Commission may prescribe for the purposes hereinbefore mentioned.

It is important to note that this type of "reopener" clause allows FERC to require a licensee like the Authority to do almost anything with respect to its reservoir management or water flow releases, whether or not such requirements are related to hydropower operations. The Authority realized that by virtue of accepting the Project license, FERC, not the Authority, would have ultimate control over a state-owned and developed water supply.

The Authority determined that such a forfeiture of legal control over an important regional water supply conflicted both with its state-chartered purposes (to develop and manage a water supply), and the terms of its bond indentures governing disposition of, and control over assets financed by publicly issued bonds. The Authority told FERC as much in a rehearing request. FERC was unsympathetic.

First, FERC determined that if the requirements of the Project license conflicted with the Authority's bond indentures, either the indentures "must be modified to authorize compliance with the license articles or the license should be surrendered or rescinded."[7] Second, as to the Authority's decision that it could not subordinate its control over water supply and releases to FERC, FERC explained that the Authority's water supply operations would be only "a relevant factor to be considered" in any future use of FERC's reserved jurisdiction under the license. Nevertheless, the FERC would retain its authority to modify any aspect of the project, dam, reservoir, and water impoundment and release regime that FERC deemed necessary to further the public interest.[8] The Authority also realized that if a future conflict arose, it could not escape FERC jurisdiction by surrendering the license, since FERC is not required to accept such a surrender.[9]

As you might imagine, the county and municipal supervisors that are charged with administering the Authority's chartered purposes and operations had no

desire to unilaterally grant to what appeared to be an uncaring and even somewhat arrogant federal bureaucracy absolute authority to tell them what to do with their dam and water supply. They also determined that even if they wanted to do so, they had no legal authority to surrender control over the region's water supply to FERC. The Authority therefore surrendered the license.[10]

While the license surrender was pending, the dual energy and environmental benefits of the Project were presented to Virginia's Congressional delegation, with the ultimate result being special federal legislation that (i) required FERC to reissue the Project's license; (ii) allowed the Authority to transfer the Project's license to a third party of its own choosing, regardless of any "municipal preference" issues; and (iii) granted the Authority the unique power to force the Project license to be terminated, and the Project removed, if FERC in the future ever required a licensee to act contrary to the Authority's obligations under its charter or bond indentures.[11] The Project license has been transferred to STS HydroPower Ltd., and the Authority retains a statutory "silver bullet" to eliminate FERC's jurisdiction under the Project license if FERC requires STS to take action contrary to the Authority's legal obligations, which include the operation and management of the Brasfield Dam and Lake Chesdin.

In order to insure Project financing, STS had to get FERC to formally recognize the Authority's "silver bullet," and to make license modifications that would make clear to Project lenders that FERC would not act in a way that would force the Authority to use its statutory protection. In August, 1992, FERC modified the Project license by strengthening special articles requiring protection of Authority operations.[12] While this order did not remove the "reopener" articles as requested, other modifications made clear that the potential for future conflict between FERC requirements and Authority prerogatives was "virtually eliminate[d].[13]

Based on our experience at Brasfield, we draw the following conclusions:

1. Hydroelectric retrofits of non-federal, publicly owned water supply structures should not require an act of Congress.

2. Hydroelectric retrofits of non-federal, publicly owned water supply structures often will require acts of Congress (or some other extreme measure) so long as FERC insists on supplanting state control over water supplies and releases by reason of any hydroelectric retrofit of such structures.

3. FERC can and should amend its licensing policies at non-federal, publicly-owned water supply structures to encourage, rather than retard, the environmentally-sound development of the latent hydroelectric potential of such structures.

II. FERC's Reserved Jurisdiction.

The foregoing should make clear that FERC's "reserved jurisdiction," as embodied in standard "reopener clauses," can create a jurisdictional conflict between state agencies charged with building and managing state water structures, and FERC, to the extent the hydroelectric potential of those state-owned structures is to be realized. FERC's reserved jurisdiction to require a licensee to amend almost any aspect of a hydroelectric project and any associated water supply structure or reservoir presents private asset owners with a continued regulatory risk of new FERC-imposed requirements. In the case of publicly-owned impoundments, that same reserved jurisdiction can become a full-blown jurisdictional conflict. The result is a very severe and continuing waste of energy at what FERC has identified as a tremendous source of unrealized hydroelectric potential: state and municipal reservoirs and irrigation structures.[14]

FERC deserves to be criticized for deliberately sacrificing hydropower retrofits of existing state-owned impoundments that could provide both energy and environmental benefits, in order to push a principal of broad federal jurisdiction in all cases. In cases like Nockamixon Hydro Associates,[15] Pennsylvania Hydroelectric Development Corporation,[16] and the Appeal Order with respect to the Appomattox River Water Authority, FERC exhibited a palpable disdain for arguments made by the public owners of water supply and impoundment structures that any hydroelectric retrofits of their structures, while welcome, had to be fully subject to the prior and primary uses of such structures. Both in cases such as these and in direct communications with concerned state agencies, FERC often has made clear that either FERC would get its pervasive jurisdiction over every aspect of the state's structure or impoundment, or no hydroelectric

development would take place. This is not a policy well designed to solicit the cooperation of public water structure owners. FERC's approach in cases like the foregoing is all the more unfortunate because it demonstrates a disregard for important federal policies incorporated in FPA Section 6 regarding certainty of license terms, policies that could be used to encourage hydropower retrofits of public water supply structures.

A typical description of FERC's view of its reserved jurisdiction is found in Pacific Gas & Electric Company, 46 FERC ¶61,249 (1989). There, FERC rejected licensee arguments that FERC could not use its standard reopener clauses to "destroy the security and effectiveness of other specific license articles" such as minimum flow provisions, and that reopener clauses were only to be used to address some "major, unexpected change." FERC explained that its broad reopener clauses indeed could be used to amend specific license articles governing particular aspects of project operation, if FERC determined at any time that such changes were necessary to help FERC achieve and maintain the comprehensive development required by FPA Section 10(a).[17]

FERC's policy of automatically requiring "reopener clauses" is troubling because it ignores the intended licensee protections of FPA Section 6 regarding licensee consent to license amendments.[18] A review of federal case law in this area makes clear that Congress has placed great emphasis on providing hydropower licensees with "fixed" license terms, so that the multiple interests involved in a hydropower development can accommodate themselves on the basis of reasonable future expectations and identifiable risks. This policy is especially important in the area of public water supply structures.

Federal case law reflects the tension between FPA Sections 6, which require a licensee's consent to a license modification, and 10(a), which mandates a comprehensive development approach by FERC. In Confederated Tribes and Bands of the Yakima Indian Nation, 746 F.2d 466 (9th Cir. 1984), the Ninth Circuit rejected a FERC argument that FERC could impose new license conditions on a recently issued license at a later date, after certain studies had been completed. The court noted that such later modifications would be limited by the protection afforded the licensee under FPA Section 6. The court explained that "[n]otwithstanding a reopener clause, FERC may not 'amend' a license in a modification proceeding without

the licensee's consent," citing FPA Section 6. The Court distinguished California v. FPC, 345 F.2d 917 (9th Cir. 1965), explaining that in that case, FERC, on the basis of prior studies, had provided for future reviews of specific license conditions, with modifications if conditions warranted. The licensee in that case was on explicit notice of areas of concern, and what might be done about them. Together, these Ninth Circuit cases might be read as allowing FERC, pursuant to FPA Section 10(a), to revisit specific aspects of a licensed project in the future to the extent those specific aspects are identified as requiring further study at the time of initial licensing. Otherwise, the licensee gains the benefit of license certainty afforded by FPA Section 6.

The D.C. Circuit has been following a similar analytical path. In Pacific Gas & Electric Company v. FERC, 720 F.2d 78 (D.C. Cir. 1983), the court was careful to stress the importance of the requirement in FPA Section 6 that licensees must agree to any license modifications. Citing legislative history, the court explained that FPA Section 6 embodies a very important Congressional policy of insuring that licenses granted by FERC promote secure license expectations. However, the Court explained that FERC could, when necessary in a particular case, shorten license terms, and require additional license conditions. This holding confirms the use of "reopener clauses" to further the statutory obligations of Section 10(a), while reminding FERC that the policy of secure license conditions embodied in FPA Section 6 is important, and is to be respected as a tool for promoting investment in hydropower development. The decision does not argue for the unthinking, automatic use of standard reopener clauses. The same can be said for subsequent D.C. Circuit decisions.[19] In fact, like the Ninth Circuit cases, these decisions appear to require an express reason, based on substantial evidence, for diminishing a licensee's FPA Section 6 protections by including reopener clauses in a license. FERC's habit of automatically including standard reopener clauses in licenses does not appear consistent with the spirit, and maybe not the terms, of these decisions.

Whatever the legality of FERC's standard reopener clauses, the fundamental lesson of the Appomattox River Water Authority cases is that imposing broad reopener clauses in licenses to retrofit publicly-owned water structures promotes neither the policies of FPA Section 10(a) nor Section 6, since no such retrofit is likely to occur under such

circumstances. It is clear that developments like the Brasfield Dam Project should be encouraged by our federal regulatory system. Yet until FERC recognizes the need to (i) respect state management prerogatives at state and municipal water structures; and (ii) protects these prerogatives through thoughtful, carefully crafted license terms, FERC will not promote the broad, balanced development policies of FPA Section 10(a), because such public structures will not be open for redevelopment. States and state agencies cannot or will not surrender control over water supply management in order to allow beneficial, if modest, hydropower retrofits to occur at their structures. And private developers will not spend years of litigation dollars attempting to force licensed projects on such structures. If there is a better policy that promotes federal/state cooperation in this area, and allows more "win-win" hydroelectric retrofits to go forward, why not adopt it?

III. A Better Policy.

In Rancho Riata Hydro Partners, Inc., 54 FERC ¶61,176 (1991), FERC hinted that it might consider a different policy regarding license terms at non-federal, publicly owned water supply structures. FERC noted that a license for a hydroelectric retrofit of a public irrigation system contained the standard Article 12 "reopener clause" described above with respect to the Brasfield Dam Project. FERC then explained that:

> Under this article, the Commission could, if it found, based on substantial evidence, that the public interest warranted doing so, insert in the license a clause subordinating project water use to specific other water uses. We do not express a judgment here as to whether future conflicts between project water use and other water rights would justify imposing a subordination clause.

54 FERC at p.61,534. The right to ask for such a "subordination" charge in the future is cold comfort to state agencies faced with open-ended reopener clauses in licenses. However, such a subordination clause provided up front as part of a new license for a hydropower retrofit of a public water structure would greatly relieve concerns over future water control conflicts, and create access to the public water structures in the first instance. In Rancho Riata, FERC confessed that it could, if it wanted to, so accommodate state water supply interests.

The projects described above in Nockamixon and Commonwealth of Pennsylvania, while determined to be in the public interest, will not be constructed. No one can be certain about how many other good projects have been bypassed, given the very real chilling effect FERC's policy has had on incentives to even assess state and municipal water supply structures for hydropower potential. The continuing waste of energy at public water structures makes a compelling case for FERC amending its licensing procedures at publicly owned water supply structures, and promoting beneficial projects and the goals of FPA §10(a) by recognizing and preserving license protections afforded by FPA Section 6. FERC already has what it contemplated in Rancho Riata; "substantial evidence" that the "public interest" will be served by protecting state water supply operations at state water supply structures.

These licensing changes need not be earth shaking. For example, in any license designed to exploit the hydropower potential of a non-federal, publicly-owned impoundment or irrigation structure, FERC could provide for the following in the license:

> Notwithstanding any other provision of this license, including any standard articles incorporated herein, the licensee shall not affect or disturb the allocation, withdrawal, impoundment, or release of water by the [public agency], and no provision of this license shall be construed as authorizing the licensee, or reserving to the Commission the authority to require the licensee, to act in a manner inconsistent with the provisions of this article.

A refined FERC policy would distinguish water structures that are owned and operated by state or other political subdivisions, and that serve a primary purpose other than hydropower use, from private water structures used incidently by public entities for water supply uses.[20] Also, FERC would not abandon its powers to set minimum flows and release regimes at private and newly constructed impoundment or diversion structures, whether public or private, and designed primarily for hydroelectric production. However, by explicitly subordinating the terms of a hydropower license to the pre-existing uses of water structures that the states and their political subdivisions built and paid for to serve other purposes (or that may be built in the future to serve such other primary purposes), FERC will open up one of the largest remaining sources of

hydroelectric generation left in the country. FERC's current policy of demanding control over state water supplies as a condition to allowing the public to benefit from hydropower retrofits of public structures is simply indefensible; it must change.

IV. Conclusion.

FERC's legislative mandate is to pursue navigation and hydropower improvements within the nation's waterways, in a balanced manner that gives effect to the various potential uses of those waterways. FERC has no mandate to preempt control over state water supplies, or to use its licensing powers as leverage to demand and obtain such control. Yet so long as FERC is willing to sacrifice largely beneficial hydropower retrofits in the name of asserting the broadest possible principals of licensing jurisdiction in every case, states will not make their water supply structures available for hydropower retrofits, and private developers and their financing institutions will not spend their time and money to attempt to force licensed projects onto such structures. FERC can better promote the balanced development mandate of FPA Section 10(a), by recognizing and promoting the licensee protections of FPA Section 6 at non-federal, publicly owned water supply structures through license terms that preserve state and municipal control over local investments in water supply structures.

[1] Partner, Rath, Young, Pignatelli and Oyer, P.A., Concord, N.H.

[2] President, STS HydroPower, Ltd., Northbrook, IL.

[3] 16 U.S.C. §791, et. seq.

[4] 16 U.S.C. §800(a).

[5] Appomattox River Water Authority, 45 FERC ¶62,243 (1988).

[6] See, 18 C.F.R. §4.30(b)(22)(defining project as any "impoundment and associated dam, intake, water conveyance facility, power plant, primary transmission line and other appurtenant facility, if . . . a man-made impoundment is used for power generation.") See also, Phoenix Hydro Corp. v. FERC, 775 F.2d 1187, 1189-1190 (D.C. Cir. 1985) (explaining that FERC could alter its prior practice of not requiring pre-existing publicly-owned impoundment structures to be part of a project's licensed or exempted property).

[7] Appomattox River Water Authority, 49 FERC ¶61,313, p. 62,175 (1989)("Appeal Order").

[8] Id., citing Trinity River Authority of Texas, 41 FERC ¶61,300, p. 61,791 (1987).

[9] 16 U.S.C. § 799 (requiring the mutual consent of FERC and a licensee to any license amendment or surrender).

[10] Appomattox River Water Authority, 53 FERC ¶62,174 (1990).

[11] Section 1075(a) of the Intermodial Surface Transportation Infrastructure Act of 1991, H.R. 776, Pub.L.No. 102-240 (1991).

[12] Appomattox River Water Authority, Order on Rehearing, Amending License and Lifting Stay, 60 FERC ¶61,083 (July 29, 1992).

[13] Id. at 1992 WL 404783, *4(FERC).

[14] According to FERC's Hydroelectric Power Resources of the United States (1988), undeveloped hydroelectric potential at state and municipal water structures constituted the largest single specific category of undeveloped potential, exceeding undeveloped potential at federal, private, utility, cooperative, and industrial structures. According to FERC, undeveloped

hydroelectric potential at state and municipal water structures exceeds 12,000 megawatts, representing over 46,600,000 megawatt hours of annual generation.

[15] 54 FERC ¶61,245 (1991).

[16] 42 FERC ¶61,070 (1988).

[17] 16 U.S.C. § 803(a).

[18] 16 U.S.C. §799.

[19] See, United States Department of Interior v. FERC, 952 F.2d 538 (1992) (FERC was justified in including reopener clauses in several licenses for "clustered" projects on the Ohio River, given the uncertain nature of water quality and fishery data).

[20] As further support for a refined treatment of publicly-owned structures, FERC might look to the licensing exemptions already accorded states and municipalities under the FPA. See, 16 U.S.C. § 828 (exempting state and municipal licensees from certain licensing requirements).

Overcoming Permitting and Licensing Inconsistencies While Developing and Operating Projects in Different Western States

Jeffrey E. Twitchell, P.E.[1]

Abstract

STS HydroPower, Ltd. (STS) is an independent hydroelectric power developer, owner, and operator of eleven small to mid-size hydroelectric projects located in five different states across the United States. STS is also actively developing numerous other projects in other states throughout the country that are in various stages of development. The projects currently being developed by STS vary from the conceptual preliminary permit stage to the final design and construction stage after issuance of the FERC license. The projects also vary significantly in licensing complexity from revitalizing existing non-operational projects to retrofitting existing dams with new hydroelectric facilities or developing entirely new projects with no existing features.

STS's development of each independent site has included either permitting, licensing, and/or obtaining license amendments; and the operation of each project has included the ongoing necessity of operating in compliance with terms and conditions stipulated in the FERC licenses by the independent local, state, and federal regulatory resource agencies.

This paper describes the inconsistent conditions placed on the developer while permitting, licensing, and operating projects in different western states. Comparisons are made of the states' and FERC's licensing consultation requirements and FERC's Section 10j process.

The different objectives, policies, and positions of the various state and regional federal regulatory agencies are also compared from a developer's viewpoint. The paper also takes a look at how private environmental licensing

[1] **Western Regional Manager - STS HydroPower, Ltd.**
7311 Greenhaven Drive, Sacramento, CA 95831 (916) 395-8811

consultants interact and negotiate differently from region to region with the resource agencies, and how they may better serve the interests of hydropower in their region.

Introduction

While developing, constructing, and operating projects in the western states of California, Washington, and Colorado, STS has noticed a significant difference from state to state on how all of the local, state and federal agencies react to the development of both conventional and small hydroelectric projects within their respective states and/or national regions. Development and operational experiences from projects located within the three states are first discussed independently and then they are compared to each other, followed by a conclusion of findings. The conclusions indicate why STS has found the state of Washington to be most difficult to develop projects in, and why California and Colorado fall respectively a distant second and third behind Washington state in local and state permitting requirements.

California

In 1988, STS constructed its 1.2 MW Kanaka hydroelectric run-of-the-river facility on a tributary to the Feather River system in the Sierra foothills of northern California; and a year later, in 1989 STS constructed its 5.0 MW Kekawaka Creek run-of-the-river facility in the Eel River coastal basin of northwestern California. The smaller 1.2 MW Kanaka project was the first entirely new project (with no existing features previously in place) that STS developed in the states after successfully rehabilitating and/or retrofitting five other small hydroelectric facilities located in Michigan and one in Colorado. Both California projects are very typical of the high-head facilities that have been predominantly proposed and constructed in the western states by independent power producers since the early 1980's.

Both projects were built on high gradient streams containing resident California fisheries; with only the larger Kekawaka Creek facility containing a very short reach of accessible anadromous fishery habitat in its lower reaches. Both projects were also favorably constructed on private lands leased to STS, with only portions of the projects' powerlines located on non-private federal lands. Both projects were acquired by STS after FERC licenses had been issued, but they both required FERC license amendments before they could be economically developed and constructed. Both projects required revising the diversion locations; and the larger Kekawaka project also required a significant revision to the penstock route and the powerhouse location. Both projects required: (i) submittal and implementation of final Erosion and Sediment Control Plans, (ii) implementation of state 401 certification waiver requirements, (iii) obtaining Corps 404 permits or a nationwide exemption,

(iv) California Department of Fish and Game (CDFG) streambed alteration permits, (v) certificates of water rights issued by the State Water Resources Control Board (SWRCB)-Division of Water Rights, and (vi) various other local approvals where required.

After obtaining cumbersome project amendment approvals STS was moderately surprised at what little interests the state regulatory agencies had in the projects conforming to the state permits after they were issued.

State representatives, after insisting upon strict erosion control and water quality standards during construction activities, visited both construction sites only once during the construction phases of each project. However, had both projects been located on public lands and fallen under the jurisdiction of the SWRCB, there may have been more of a heightened interest during project construction and operation, as is evident on other similar operational projects located in California. Although not all hydro projects in California require state appropriative water rights, they all do require a host of state certifications as noted in Table 1 from the SWRCB. Some of the state authorizations are obviously required before issuance of a FERC license, and others are a requirement before a project becomes commercial operational.

TABLE 1

STATE OF CALIFORNIA
WATER RESOURCES CONTROL BOARD (S)
Types of Authorization

- Clean Water Act 401 Certification
- Certification to California PUC that Project has either Appropriative Water Rights, or Riparian Water Rights or Pre-1914 Water Rights
- Certification to California PUC that Project has a valid 401 Certification, or a valid 401 Waiver
- National Pollutant Discharge Elimination System Permit (NPDES) Stormwater Permit - similar to FERC Erosion and Sediment Control Plans

Before the California SWRCB can issue a 401 certification or an appropriative water right, the project proponent must have the SWRCB prepare or approve a California Environmental Quality Act (CEQA) document (1. California, 1989). Collective preparation and approval of the CEQA document and issuance of the state permit should not take more than 12 months according to state stature, but 12 to 24 month delays beyond the initial 12 are quite common and are expected. This delay in most cases is acceptable provided the CEQA process is implemented early on in the FERC license application process which typically exceeds three years. However, a major conflict almost always occurs between the California state 401 process and the FERC licensing process.

The project proponent is encouraged to consult with the SWRCB early in the FERC process, but encouraged not to officially apply for a 401 certification and/or waiver until immediately before filing its license application at FERC. The 401 request is postponed because the state acknowledges that the SWRCB cannot always act on a project's 401 request until after a year of review; and if the CEQA document is incomplete the SWRCB will almost always dismiss each project's 401 certification request without prejudice. The state recognizes that two 401 certification denials by the state will trigger an automatic denial of the FERC license by FERC; and thus suggests the applicant and the SWRCB jointly file for an extension of time or a stay of time at FERC before FERC lets the one year clock expire yielding an automatic 401 waiver. If after nine months into the CEQA process the SWRCB determines it cannot issue a 401 certification in conformance with the state's one-year CEQA requirements the stay at FERC is usually requested.

If a project developer was insistent upon completing a CEQA document within 12 months and could show FERC that the SWRCB was at fault in delaying the 401 certificate, the certificate could be deemed waived by FERC and a FERC license issued; but independent of FERC's conclusions, before commencing commercial operations the State Public Utilities Commission (by law effective January 1933) would need certification from the SWRCB that the project was in conformance of the 401 certification and CEQA processes as administered by the state. Before commercial operation the state PUC would also need certification from the SWRCB as noted in Table 1, that the project possessed either an appropriate water right, a riparian water right, or a pre-1914 water right.

It should be noted that the project applicant, unless it is a county or municipal water district recognized by the state, the SWRCB is the only agency within the state that can act as the lead CEQA agency in processing both a 401 certificate and an appropriative water rights application for hydro

projects. In previous years, the county in which a private project was to be located could act as the lead CEQA agency. This is no longer the case.

California has no state comprehensive hydro resource plan but it has a collection of specific state and/or federal wild and scenic river sections that prohibit the development of hydroelectric plants; the state strongly discourages projects in anadromous reaches but does not automatically prohibit future development in anadromous zones as it is done in the northwest, particularly in the state of Washington.

Washington State

While STS was completing its two high-head projects in California, STS was presented with numerous other high-head, small hydro opportunities in the state of Washington. Along with a seemingly abundance of renewable water resources in Washington state, STS quickly learned that each project required numerous overlapping and sometimes conflicting regional, state and local permit approvals before construction could proceed on projects that had already been issued FERC licenses. In addition to a higher number of state and local permits, it was also readily apparent that the regional agencies placed a much higher value on the natural resources in the four northwest states than in other regional areas of the country. Relative to other western states, stricter ramping rates, tighter fish screening requirements for either resident and/or anadromous fisheries are recommended by the state of Washington. A higher public value is also placed on the natural resources normally utilized by hydro sites because of the presence of Washington state's large coastal commercial fishery, the close proximity of hydro resources to major metropolitan areas, and the public's awareness of the large scale fishery impacts that have been associated with Bonneville Power Administration's (BPA) Columbian River basin projects.

To protect the natural resources of the state from allegedly being overdeveloped or significantly impacted individually or cumulatively with both small and large hydro facilities alike, the Washington State Department of Ecology (WDOE), in cooperation with other state agencies and local county governments, have gone to great lengths to develop "tools or weapons" (2. Sakrison, 1992) in the form of local permit approvals that now must be obtained by project proponents before, during, and/or after the issuance of a FERC license. Project proponents in Washington state are primarily held hostage to all of the state's recommended conditions through the following mechanisms: (i) the state's own interpretation and implementation of the Clean Water Act (CWA) 401 certification process; (ii) the state's own Washington State Hydro Development/Resource Protection Plan – a state regional plan forwarded to FERC with an effective date of January 1993; and (iii) the state's interpretation and enforcement of the Coastal Zone

Management Act (CZMA) of 1971 that the Federal Secretary of Commerce delegated to the states to administer prior to the PURPA legislation of 1978.

The Washington state legislature had good intentions when it directed the State Department of Energy to develop a statewide hydroelectric plan in 1990 (RCW 90.54.800) that could be sent to FERC for consideration and acceptance as a regional comprehensive development plan that was to balance natural resource values with potential renewable, hydroelectric energy resource values. The legislature's plan was also intended to enhance the existing hydropower permit review process. Unfortunately, an unbalanced resource plan that serves to further hinder the local permit process was developed primarily by the Department of Ecology who placed significantly higher values for natural resources over the viable hydro resources located within the state. The state plan identifies and categorizes natural resources into three separate areas: sensitive/hydropower opportunity areas; less sensitive/hydropower opportunity areas; and resource protection areas. The state's resource plan is also an expansion of the Northwest Power Planning Council's (NPPC) list of protected areas and thus fewer sites are now available for hydro development as previously identified by the NPPC.

Table 2 below shows that only 44% of the resources under present development within the state of Washington would be found acceptable by the State Hydro Development Resource Plan which became effective January 1993 (4. Washington State, 1992).

Table 2
Washington State Hydropower Development/Resource Protection Plan (effective January 1993)

Active FERC Projects	Resource Classifications/ (FERC status)	MW's	aMW's	% of Total MW's
44	Hydro Opportunity Reaches Sensitive & Less sensitive	357	179	44
28	Resource Protection Areas (FERC Stage 1 completed)	253	126	31
18	Resource Protection Areas FERC Stage-1 uncompleted	202	101	25
90	Total Active Projects	812	406	100%

The state's interpretation of the federal (CZMA) as implemented by the state's own Coastal Zone Management Plan (CZMP) could be more threatening or detrimental to a good portion of the projects in the state of Washington, particularly the projects located in the 15 western counties that are partially bounded by a salt water shoreline.

The state CZMP, as it was implemented in the early 1970's, was never intended to prohibit hydroelectric development, but it didn't contemplate non-federal hydro development with the later implementation of PURPA in 1978. The state CZMP is interpreted by WDOE to preclude hydro because hydro development was not identified as a potential use below the high water line of natural water courses containing average annual flows of 20 cfs or greater.

The WDOE believes that the FERC staff should view the state CZMP in the same light as the state 401 certification process (3. Shorin, 1992), and in response to the WDOE's position on the CZMP, FERC staff has suggested, and in some cases has requested, some project applicants to obtain certification from the state and/or its respective counties that the proposed projects are in compliance with the state CZMP before FERC will issue a license.

The State Environmental Protection Act (SEPA) and the CZMP are primarily administered by the counties and not the WDOE. This unfortunately has set up a "Catch-22" situation where county and local regulations almost always require full approval from other federal and state agencies <u>before</u> issuance of the CZMP permit. However, in the event a CZMP conformance certification is issued with other local permits prior to the issuance of a FERC license, the project proponent still runs the risks of the WDOE or others challenging the county's findings, and the local short-term permits expiring before the license is issued or before construction begins two to four years after the FERC license is issued. If the WDOE and the counties strictly enforce the CZMP it could impact and/or significantly delay as much as 60% of the 812 MW's identified in the State Resource Plan. This represents 70% or 63 of all the 90 projects proposed in the state of Washington as of January 1993. This represents over 50% of the small hydro projects identified by the NPPC's 1991 Conservation and Electric Power Estimates.

In addition to the accumulation of the state approvals required within the state of Washington and other adjoining northwest states, one should note if a project proponent decides to sell the project output to the Bonneville Power Administration (BPA) after receiving a FERC license, the project will require the preparation and public notice of another Environmental Assessment or EIS in addition to the one already published by FERC. BPA

is of the opinion that the long term purchase of a licensed hydro resource, constructed or unconstructed, puts BPA in the lead as the federal lead agency responsible for adhering to NEPA.

This condition places additional development risks on the project proponent, particularly if environmental conditions change and/or additional species in the Northwest become listed as threatened and/or endangered. Although the project may not impact such species, the project may contain habitat suitable for the subject species, and additional studies may be called for by BPA to further confirm that no impacts will occur. BPA's review will obviously add further delay to projects beyond the lengthy review process that already exists in the Northwest region. The long term impact could be higher avoided cost rates passed on to the electric consumers in the northwest.

Colorado

While developing two small hydroelectric projects in the state of Colorado (one 2.5 MW retrofit project at the Sugar Loaf Dam, outside of Leadville and one 4.5 MW run-of-the-river facility for the Town of Telluride on the San Miguel River), STS found little resistance from the state in the form of state regulations that would impede project development. The state in both cases was very responsive and easy to work with while obtaining construction approvals and FERC license conditions for both projects that utilized both federal and private lands.

Only three key state agencies were involved in providing comments and recommendations for the projects located in Colorado. The Colorado Department of Health administered and/or waived 401 certifications for the projects without the prerequisite of preparing state environmental documents. The Colorado Department of Wildlife (CDW) set terms and conditions to protect the fish and wildlife resources in the project areas, and the Colorado Water Conservation Board (CWCB) decreed water rights with proper public notice for both projects without much delay. Fortunately for the run-of-the-river facility, the CDW had previously established an instream flow decree with the CWCB for the San Miguel River in conjunction with a statewide wildlife conservation plan. The decreed instream flows were found acceptable and therefore no negotiations were required to agree upon an acceptable instream flow that would become a condition of the FERC license.

Consultation Requirements - and the FERC 10j Process

The FERC 10j process is a vehicle that FERC devised to expedite resolution of resource agency preferred recommendations and studies that the project applicant may not concur with during the licensing phase. Of the three western states mentioned above, the 10j process will be most often used

and needed by developers in Washington state and in similar states where the state(s) and federal agencies, inclusive of the Native American Indian Tribes, commonly ask for multiple studies and extra time to develop project comprehensive conditions.

The 10j process over time should prove most effective when properly consulting with the Indian tribes, as they have been known to change or delay their formal comments or policy positions affecting the development of projects. The 10j and formal consultation process will keep the tribes more focused with the specific issues related to specific projects versus other general tribal issues that surface while discussing the subject of hydroelectricity.

Observations

When STS has asked environmental consultants in the northwest to either scope or perform environmental studies STS has consistently found the consultants in the region to be very passive and less supportive in comparison to other regions when defending the project proponents' positions on common environmental issues or studies. The private environmental consultants in the northwest do not seem to challenge the agencies' requests or recommendations as much as consultants do in other regions. Rather than confronting the agencies regarding questionable study requests, the consultants seem more content pleasing the agencies by performing additional studies, thus requiring the project applicants to invest more time and resources. As a result, a good number of the applicants in the State of Washington are serving as deep pocket environmental research centers for the regulatory resource agencies. The environmental consultants of the northwest would better serve the interests of the hydropower industry in the region if they challenged the agencies more. If the agencies are not challenged by the environmental consultants and developers alike there will continue to be a large imbalance of higher values placed on natural resources over clean renewable hydroelectric resources.

Conclusions

STS has found the western states with the largest number of potential hydroelectric resources to be the same states with the largest number of state regulations and compliance recommendations that can significantly curtail and block the development of the preferred renewable resource. If given a choice to develop one of five identical projects in five different states, all of which contained similar electrical generation benefits and similar environmental impacts, STS would recommend developing the project in the state with the fewest amount of hydro resources. STS would also suggest developing and operating projects on private property with fewer regulations over projects situated on federal and/or state lands.

References

1. California State Water Resources Control Board, A Guide to California Water Right Appropriations, (January 1989).

2. Sakrison, Rodney G., WDOE, State Water Rights and Section 401 Process - Is This Process a Tool or a Weapon? (January 1992).

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4. Washington State Energy Office, Washington State Hydropower Development/Resource Protection Plan, (December 1992).

The Curse of Sisyphus: Using License Reopeners to Impose Additional Environmental Conditions on FERC Licensed Projects

Michael A. Swiger
Miriam S. Aronoff[1]

I. ABSTRACT

The Federal Power Act's ("FPA's") licensing provisions were intended to encourage development of hydroelectric resources by providing security of investment. Federal Energy Regulatory Commission ("FERC") practice and court decisions have helped erode this security of investment through the use of license "reopeners" to impose additional environmental conditions on licensed projects. License reopeners have also been used to impose temporary environmental conditions on projects pending relicensing. This paper examines different types of reopeners and the implications of their use.

II. INTRODUCTION

The Federal Water Power Act of 1920 ("FWPA"), the FPA's predecessor statute, established a national policy for water power development and vested exclusive licensing authority in the Federal Power Commission (now FERC). One of the principal features of the FWPA was that licenses would be issued for a renewable term of up to 50 years, with no alteration of the license terms except by mutual consent of the licensee and FERC. Section 6 of the FPA provides that license terms and conditions "shall be expressed in [the] license" and that the terms "may be altered...only upon mutual agreement between the licensee and the Commission." One of the main purposes of this provision was to provide security of investment for developers of hydroelectric resources.

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[1]Michael A. Swiger is a Member, and Miriam S. Aronoff is an Associate, of the law firm of Van Ness, Feldman & Curtis, A Professional Corporation, 1050 Thomas Jefferson Street, N.W., Washington, D.C. 20007.

Despite this original intent, FERC has for many years included "reopener" provisions in its standard license articles which allow FERC to require changes to licensed projects in the interest of wildlife, navigation, recreation, and other factors. FERC has also increasingly burdened licenses with special reopeners imposing post-licensing study or monitoring requirements, which can result in additional environmental mitigation requirements. These types of reopeners are arguably inconsistent with Congress' intent, as expressed in FPA Section 6, to guarantee secure and certain license conditions.

FERC rationalizes the inconsistency between use of reopeners and Section 6 by asserting that the applicant "consents" in advance to reopener requirements at the time that the license is issued. The applicant's alternative to giving its "consent" to unknown conditions is, of course, not to accept the license.

While license reopeners allow for changes in circumstances, they also create uncertainty as to what terms and conditions will have to be complied with over the course of the project license. Like the mythical Sisyphus, the licensee slowly and painfully rolls the rock up the hill of license compliance. As he nears the peak of compliance, the reopener is invoked and new conditions are imposed. The licensee, like the Corinthian king, will watch helplessly as the rock tumbles down to the bottom of the hill. He then will comply with the new conditions, rolling the rock up the hill until the next time that the reopener is invoked and the process begins all over again.

III. STANDARD LICENSE ARTICLE REOPENERS

The standard license articles contain a number of reopeners that reserve FERC's authority to include later conditions on hydropower projects. Standard reopeners include the right to require, after notice and opportunity for a hearing: changes in stream flow measuring devices;[1] installation of additional capacity or other changes in the project;[2] coordination of operations with other projects;[3] reasonable use by third parties of the licensee's reservoir or project properties;[4] and reasonable modifications for the benefit of fish and wildlife resources[5] and for recreational benefit.[6]

There are limits on FERC's ability to impose additional conditions unilaterally. First, the standard reopeners specify that requirements will be imposed only after notice and opportunity for a hearing. If there are disputed issues of fact, an evidentiary hearing may be called for. Second, some reopeners make clear that the requirements must be "reasonable." A modification that resulted in a severe diminishment of the project's value or made it economically infeasible to continue certainly could be challenged as unreasonable. The standard article which reserves FERC's authority to require the installation of additional capacity or "other changes" in the project, specifically notes that the

changes ordered by FERC must be "economically sound."[7] Courts have upheld FERC's inclusion of standard reopeners.[8]

FERC has construed the standard reopener requiring a licensee to install additional capacity to apply only to the licensee and not to permit third party encroachment on the license.[9] It has also permitted deletion and modification of standard license reopeners when they conflicted with the primary purpose of a water supply project.[10]

FERC has used a standard reopener, requiring the coordination of the operation of the licensee's project with other projects, to solve a dispute concerning two hydraulically interdependent hydroelectric projects. In the October 8, 1992, Philadelphia Corporation v. Sandy Hollow Power Company proceeding,[11] FERC found that the two projects encroached on each other due to inaccuracies in the survey data at the time of licensing. It used the standard reopener, present in both licenses, to amend the licenses and resolve the problem. FERC "split the difference" economically so that each project would incur an equal loss of generation of electricity.

IV. SPECIAL REOPENERS

In addition to standard reopeners, FERC has increasingly used reservations of authority to address particular issues that arise in the course of a licensing proceeding. They generally involve questions regarding the appropriate scope of environmental mitigation. Special reopeners typically require the licensee to submit a mitigation plan after consultation with resource agencies. The plan is then subject to FERC's modifications. Special reopeners have been used to address a variety of issues such as: structural improvements for aquatic habitat and fish passage; wetlands enhancement; management of habitat for endangered and threatened species; and impacts on dissolved oxygen and water temperatures. Resource agencies generally play an important role. The agencies submit comments and recommendations to FERC and the licensee must either incorporate them in the plan or explain why they were not incorporated. Consultation with the resource agencies often continues into the implementation phase of the plan.

These types of special reopeners have been criticized by agencies and environmental interests as deferring consideration of important adverse environmental impacts until it is "too late" (i.e. the license has been issued). However, the practice has been upheld by the courts.[12] FERC has stated that it will continue to include license conditions requiring further studies because these studies enable FERC to "assess the effectiveness of mitigation measures; to fine-tune project facilities and operations; to secure information that cannot be obtained prior to license issuance; or to address new circumstances that may arise in the future."[13] As discussed above, inclusion of a reopener to address

"new circumstances" is arguably inconsistent with FPA Section 6. Licenses have limited terms, not unlimited terms. Changes in circumstances can be addressed at the expiration of the license term on relicensing.

The advantage of special reopeners to applicants is that they allow licenses to be issued sooner than if FERC required every licensing issue and dispute to be definitely resolved prior to issuance of the license. In truth, however, these types of reopeners often allow FERC to avoid tough licensing decisions. Agencies and environmental interests almost always claim more study of environmental impacts is needed. It is easier for FERC to require post-licensing studies, with final mitigation measures to be decided at a later date, than to make the decision that "enough is enough." Agreeing to post-license studies can be a tempting trap for applicants. By deferring decision on critical issues, FERC imposes hidden costs and uncertainties on the licensee. The deferred costs of monitoring, studies, and implementation of the plans may result in the project becoming less desirable, or even economically infeasible.

V. THE PLATTE RIVER CASE: ANNUAL LICENSE REOPENERS

Under Section 15(a) of the FPA, on expiration of a license, FERC is required to issue an annual license under the terms and conditions of the original license until a new license is issued. In Confederated Tribes and Bands of the Yakima Indian Nation v. FERC,[14] the court stated in dictum that FERC has the authority to issue annual licenses with interim environmentally protective conditions if the original license contains a reopener. The legislative history of the Electric Consumers Protection Act of 1986 ("ECPA"), which amended FERC's relicensing procedures, cited to the Yakima decision for the proposition that annual licenses can include fish and wildlife protection provisions.[15]

In Platte River Whooping Crane Critical Habitat Maintenance Trust v. FERC,[16] the United States Court of Appeals of the District of Columbia Circuit found that FERC abused its discretion by refusing to undertake any inquiry into the need for environmentally protective interim conditions for threatened and endangered species in the annual licenses. It remanded the case back to FERC so that it could explore the need for interim "rough and ready" protective measures. The court reasoned that this was necessary because it would take several years for the environmental issues involved in the relicensing proceeding to be resolved. FERC had denied the petition of one of the parties in the relicensing controversy, the Platte River Whooping Crane Critical Habitat Maintenance Trust ("Trust"), to determine the need for interim environmentally protective conditions. FERC had based its decision on the fact that there was no substantial evidence on which to determine appropriate mitigative conditions and that the necessary information would be developed and addressed during the relicensing proceeding.

There were two project licenses at issue in the case. The court noted that FERC could unilaterally impose interim conditions on the Nebraska Public Power District's ("NPPD's") North Platte Keystone Diversion Dam Project annual license because its original license contained reopener provisions in which FERC reserved the right to impose conditions for the protection of beneficial uses.[17] The court stated that, although the Central Nebraska Public Power and Irrigation District ("Central") license for the Kingsley Dam Project did not contain similar reopener provisions, FERC could seek Central's consent to voluntarily amend its license.

In its rulemaking to implement the relicensing provisions of ECPA, FERC modified its rules specifically to conform with the court's opinion in Platte River. New Subsection 16.18(d) states that, when issuing an annual license, FERC may incorporate additional or revised interim conditions if necessary and practical to limit adverse environmental impacts.[18] FERC has acknowledged that implicit in the rule is the prerequisite that FERC had the appropriate reservation of authority in the expired license, and therefore can unilaterally impose interim conditions in the annual license.[19] FERC also has acknowledged that it must base interim conditions on substantial evidence.[20]

The Platte River case illustrates two fundamental problems with the imposition of interim conditions on annual licenses. On remand from the D.C. Circuit, FERC determined that there was a need for interim measures to prevent potential harm to certain endangered and threatened species. However, this was decided based upon an admittedly incomplete factual record.[21] The consequences were far-reaching and severe. FERC required 2000 cfs minimum flows for whooping crane habitat in the spring and fall based on an analysis which purported to show that providing such flows would have no adverse impact on irrigation water supply. Only three months after issuing its order imposing the conditions, FERC was forced to suspend the flow requirements because of the significant depletion of NPPD's irrigation water for the entire year.

Significantly, NPPD and Central had requested an evidentiary hearing prior to imposition of interim conditions. They asserted that there were serious flaws in the flow models put forward by the Trust and resource agencies--on which FERC ultimately relied in imposing the interim minimum flow conditions--which would be uncovered in a formal hearing with discovery and cross-examination of witnesses. FERC denied the hearing request, claiming that since it only was imposing "rough and ready" interim measures, a "paper hearing" was sufficient.

The second problem suggested by the Platte River case is that once interim measures are placed in the annual license, FERC may view them incorrectly as part of the "baseline" for permanent relicensing conditions. Interim conditions become a starting point for debate in the relicensing

proceeding even though the interim conditions may be completely inappropriate as long term conditions. The danger is that such conditions may not receive an appropriate level of scrutiny in the relicensing proceeding because they already are "accepted" and part of the project operation.

VI. CONCLUSION

Hundreds of projects are up for relicensing at FERC. One hundred sixty five licenses are due to expire by December of 1993. Many of the relicensing proceedings will involve environmental controversy. Although interim license conditions have not been widely used to date, it is likely that there will be increased pressure from agencies and environmental groups to follow the Platte River example and to include conditions in annual licenses. This is particularly true as it becomes clear that the relicensing process for many of these projects will be lengthy.

A licensee faced with a reopener proceeding should consider two strategies. First, the licensee should consider proposing an expedited evidentiary hearing. An expedited evidentiary hearing allows both the licensee and other parties to challenge the assumptions of relevant studies. This will lead to better decision-making by FERC. Agreeing to an expedited hearing may alleviate some of FERC's concerns about short-term environmental impacts, particularly with annual licenses.

Second, the licensee may want to consider proposing its own voluntary modifications. The licensee should be able to justify the conditions that it is proposing with reliable factual information. In the case of annual license conditions, it should take care to distinguish short-term measures from the long term conditions that may apply in the new license. Finally, accurate records of interim mitigation measures may be useful in demonstrating the appropriateness or inappropriateness of proposed long-term conditions.

ENDNOTES

1. 54 F.P.C. at 1802 (1975). For example, see Article 8 in Form L-1.

2. Id. For example, see Article 9 in Form L-1.

3. Id. For example, see Article 10 in Form L-1.

4. 54 F.P.C. at 1803 (1975). For example, see Article 13 in Form L-1.

5. 54 F.P.C. at 1804 (1975). For example see Article 15 in Form L-1.

6. Id. For example, see Article 17 in Form L-1.

7. 54 F.P.C. at 1802 (1975). For example, see Article 9 in Form L-1.

8. For example, in U.S. Dept. of Interior v. FERC, 952 F.2d 538 (D.C. Cir. 1992), the court held that FERC acted reasonably in the face of uncertainty by addressing unknown environmental impacts postlicensing through a standard license reopener clause and postlicensing conditions.

9. Pacific Gas & Electric Co. v. FERC, 720 F.2d 78, 89 n.27 (D.C. Cir. 1983) citing North Kern Water Storage Dist., 16 FERC ¶ 61,082 (1981). See also Carry Falls, 41 FERC ¶ 61,069 (1987). In Pacific Gas & Electric, however, the court found that there was a "de minimus" exception to Section 6 that permitted a reduction of 0.3% in generating capacity. This encroachment was considered too insignificant to be considered an alteration under Section 6.

10. Appomattox River Water Authority, 60 FERC ¶ 61,083 (1992). In this proceeding, FERC partially granted the request of the Appomattox River Water Authority to amend its license to delete standard license articles. Two standard articles that FERC deleted concerned the coordination of operations of the project with other projects and the permission of reasonable use by third parties of its reservoir or project properties. FERC stated that there would have been little chance of its invoking these two reopeners in light of the primary use of the water for municipal water supply.

11. 61 FERC ¶ 61,045 (1992).

12. U.S. Dept. of Interior v. FERC, 952 F.2d 538 (D.C. Cir. 1992).

13. 55 Fed. Reg. 4 (January 2, 1990), FERC Statutes and Regulations ¶ 30,869 at p. 31,614.

14. 746 F.2d 466 (9th Cir. 1984), cert. den., 471 U.S. 1116 (1985).

15. H.R. Rep. No. 99-507, 99th Cong., 2d Sess. 33 (1986).

16. 876 F.2d 109 (D.C. Cir. 1989).

17. NPPD's Article 7 provided that the "licensee shall take such measures as may be necessary or desirable for the efficient operation of the project...or for such other purposes as may be directed by the Commission." NPPD's Article 12 stated that the "United States specifically retains and safeguards...the right to require operation by the licensee of the project works of the project, so far as such operation involves the use, storage, and discharge from storage of waters affected by this license, under such rules and regulations as the Federal Power Commission may prescribe for the protection of life, health, and property...for irrigation and for other beneficial public uses, including recreational purposes."

18. 18 C.F.R. § 16.18(d) (1989).

19. Central Nebraska Public Power and Irrigation District, Nebraska Public Power District, Order on Interim License Conditions ("February 14 order"), 50 FERC ¶ 61,180 at p.61,530, citing 50 FERC ¶ 61,241 (1990).

20. 50 FERC ¶ 61,180 at p.61,527 (1990).

21. Id. at p.61,532. "...information gaps remaining in the record complicated our assessment of the need for interim conditions." Id.

An Overview of the Federal Headwater Benefits Program

Charles K. Cover, P.E., M. ASCE[1]

ABSTRACT

When Congress enacted the Federal Water Power Act (Act) in 1920, it wisely included provisions to compensate the owners of headwater improvements for benefits provided to downstream projects. The benefits Congress envisioned are those of increased energy production at hydropower plants resulting from river regulation provided by large storage reservoirs. Section 10(f) of the Act authorizes the Federal Energy Regulatory Commission (Commission) to determine the amount of energy benefits provided by headwater projects. In addition, the Act requires the Commission to collect assessments for energy benefits provided by federal improvements for reimbursement to the U.S. Treasury.

Managing the headwater benefits program has been challenging from a regulatory perspective. In the early years of the program, determining energy gains received at hydropower projects was a time-consuming task, accomplished without the aid of computers to analyze the many years of numerical data required to complete a study. The Commission developed a headwater benefits computer model in the 1970's to calculate energy gains by simulating operation of downstream projects with and without the effects of the headwater project. While the computer model has increased the number of investigations completed, technical and administrative obstacles have delayed the collection of assessments.

To date, the Commission has completed investigations of more than 61 river basins throughout the country and collects approximately $6 million

[1] Civil Engineer, Federal Energy Regulatory Commission, Office of Hydropower Licensing, Division of Project Compliance and Administration, 810 First Street, N.E., Washington, D.C. 20426

annually in energy gain assessments for the U.S. Treasury. A reassessment of the program is underway and could result in up to 40 additional river basins that require detailed investigations to determine the amount of energy gains present.

This paper discusses the history of the headwater benefits program and the methodology used to calculate energy gains. It also examines several technical and administrative issues the Commission has faced over the years and discusses the future direction of the program.

INTRODUCTION

Hydropower represents 10% of the electric energy produced in the United States. Its potential as a low cost renewable energy resource will continue to make it attractive. Unlike fossil fuels, which are depleted when used, falling water performs the task of generating power over and over. Depending on the physical characteristics of a river system, the surrounding topography, and man's efforts to harness this resource, the power of flowing water can benefit multiple hydropower projects. Benefits accrue in the form of increased hydropower production by the regulation of a river basin by upstream projects.

The Federal Power Act(1) addresses the benefits of water as a renewable resource. Section 10(f) of the Act requires licensees who benefit from the construction of another licensee, permittee, or the United States to compensate the owner of the upstream storage reservoir or other headwater improvement. In 1935, the Act was amended to allow for the collection of headwater benefits charges from unlicensed hydropower project owners who receive additional energy production benefits.

Hydropower project owners that benefit from headwater improvements are required to reimburse the headwater project owner for an equitable part of the annual interest, maintenance, and depreciation expenses. Hydropower project owners are assessed headwater benefits charges based on the additional energy production provided by regulated flows from the upstream project, as well as its apportioned power cost. The Commission determines what the headwater benefits payments are by conducting an investigation of river flows and energy generation in the river basin. Energy gains are the difference between the energy a downstream project would produce with the headwater project and the energy it would produce without that project. The cost of the headwater project is based on the project costs attributable to facilities that provide power benefits.

PROGRAM BACKGROUND

Most of the work done in the early years of the Commission was directed toward river basin studies and supporting the overall development of hydropower resources throughout the country. It wasn't until Congress amended the Act in 1935 that the Commission conducted its first headwater benefits (HWB) investigation. In conjunction with the 1935 amendment, the Commission held a comprehensive hearing to discuss project development in the San Joaquin River Basin in California. At issue was what contribution should be made by hydropower users to licensed upper storage developers for energy gains made possible by their development. This investigation, which involved non-federal projects, lasted five years and resulted in the Commission's first HWB determination in 1939. While the Commission noted in its finding that additional headwater benefits cases were sure to follow, World War II and the nation-wide need to find additional sources of electric energy and insure their efficient development delayed additional investigations for over 10 years.

The Commission's first HWB investigation to recover costs for energy gain benefits provided by federal projects was started in 1950 and involved the Grand Coulee project in the Columbia River Basin. As new storage reservoirs and hydropower plants came on-line to meet the nation's energy demands of post-war growth, new HWB investigations were initiated shortly thereafter. By the end of the 1950's, the Commission had 15 investigations underway and had collected approximately $200,000 for energy benefits provided by federal projects.

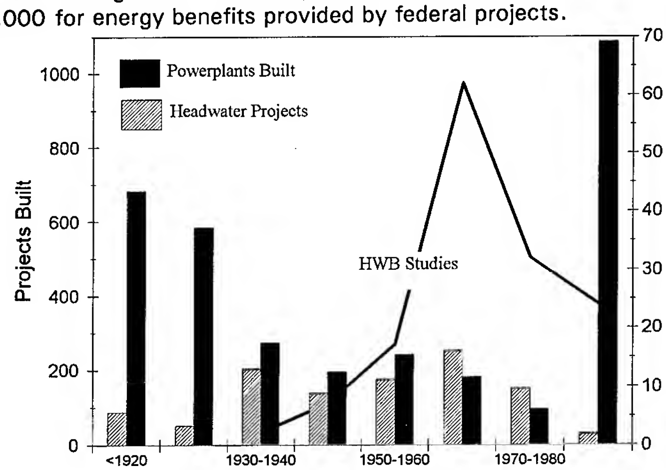

Figure 1 - Projects vs HWB Studies

To date the Commission has investigated headwater benefits in more than 61 river basins throughout the country. These investigations resulted in the collection of $215 million which has been returned to the Treasury to offset the cost of providing energy gain benefits to downstream hydropower plants(2). Figure 1 shows the number of reservoirs and hydropower plants constructed in 10-year increments in relation to Commission river basin investigations for headwater benefits.

INVESTIGATION PROCEDURES

A headwater benefits investigation is undertaken when the Commission believes that an upstream storage reservoir regulates river flows and provides downstream hydropower plants with the benefit of increased energy production otherwise not available. HWB investigations are governed by the procedures outlined in the Commission's regulations(3). There are four steps involved in a detailed investigation: Preliminary assessment, detailed review, energy gain calculations, and cost apportionment.

Preliminary Assessment

The gathering of data in the river basin is the first step in determining the potential energy gains received at hydropower projects. This information includes:

1. River basin planning reports;

2. A listing of all reservoirs and their storage capacity, and powerplants and their installed capacities; and

3. Project data, including operational histories, reservoir storage changes, and powerplant generation data.

From this information, a decision is made whether the basin warrants a more detailed review. This decision is based in part on the number of projects in the basin, the storage capacity of headwater projects, and the generating capacity of downstream hydropower plants.

Detailed Review

Specific project information is needed to determine the level of investigation and the potential magnitude of energy gains provided by headwater projects. Additional data is gathered for the river basin under consideration, including:

1. River basin hydrologic data; and

2. Cost data for federal headwater improvements.

To get a feel for whether there are energy gains present in a river basin, it is desirable to review 10 to 15 years of records to compensate for wet and dry water years, or powerplant outages that could affect energy gains. Stream gauge records are reviewed in conjunction with powerplant generation data to identify trends in additional energy generation that could result from upstream project regulation. After all data has been assembled, a decision is made on the level of investigation necessary to determine energy gains.

Energy Gain Calculations

There are three levels of HWB investigations. The appropriate level depends on several factors, including the complexity of the river basin, river hydrology, data availability, and estimated energy gains.

LEVEL 1 Flow Duration/Generation Analysis Based on identified flow and generation trends from statistical data analysis, annual energy gains at downstream hydropower plants are estimated.

LEVEL 2 Monthly Data Analysis An analysis of the river basin is conducted using monthly data to ascertain energy gains from the regulated flows provided by headwater projects.

LEVEL 3 Daily Analytical Analysis A river basin analysis conducted using a mathematical model to simulate daily operation of all headwater storage projects and their combined effect on energy production at downstream hydropower plants.

Cost Apportionment

The Federal Power Act specifically states that only a portion of those headwater project investment costs associated with power benefits may be recovered through headwater benefits assessments. These costs include interest, depreciation, and maintenance for the headwater storage project. The Commission's cost of the investigation can also be recovered. Headwater project investment costs are generally provided by the federal agency which owns the storage project. The federal

government has used various methods to calculate its investment cost among the various project purposes(4). The current method of cost allocation is known as the "separable costs-remaining benefits method." In this method, the separate cost for each project function (i.e., power, irrigation, flood control, etc.) is determined in relation to the energy benefits provided downstream.

CALCULATING HEADWATER BENEFITS

Until the mid 1970's, conducting a headwater benefits investigation was extremely time consuming because of the unavailability of a computer mathematical model to evaluate the years of data required to compute energy gains. Each year's records had to be painstakingly reviewed to evaluate hydrologic trends, irregular data, new projects that had an effect on the river system, and other occurrences that might affect energy generation. The Commission's desire to obtain an estimate of the actual extent to which downstream projects had benefitted by headwater improvements, combined with the increasing complexity of river basin systems and their intense development, generally resulted in a one- to five-year time frame to complete each investigation.

These early HWB investigations were conducted by using various methods to compute the energy gains. They included: preliminary assessment settlements, coordination agreements, critical period[2] energy calculations, and energy calculations based on the monetary value of power benefits. The value method to compute energy gains was abandoned in the 1970's because of its sensitivity to fluctuating energy prices.

The Commission began using the energy gains method in its investigations to achieve several goals:

Apportion the 10(f) costs based on actual energy gains received;

Adopt an easily administered factual methodology for calculating energy gains;

Provide developers with a greater degree of predictability; and

Reduce the costs of HWB investigations.

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[2] "Critical period" means the time during which all water storage at a reservoir would be released for power production, assuming recurrence of the adverse streamflow conditions of record for the area.

A mainframe computer model was developed to evaluate the river basin under various river flow scenarios. The model, known as the Headwater Benefits Energy Gains (HWBEG) Model(5), determines the energy gains by calculating the theoretical downstream hydropower plant energy production with each regulating headwater project removed from the river basin, and comparing the results with actual plant production with the headwater project in place.

The HWBEG Model was developed using the PL/1 computer language, which is similar to FORTRAN and other high level programming languages. The program is able to represent a river basin hydrologic network by nodes and branches as shown in Figure 2.

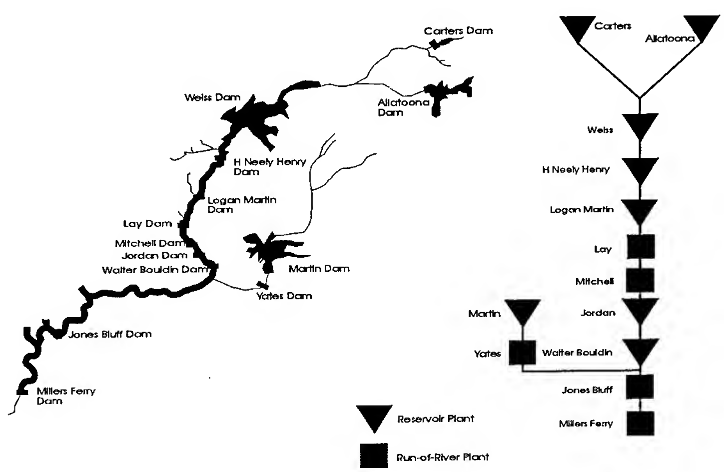

Figure 2 - River Basin Network and Computer Schematic

Pertinent river basin and project parameters, such as lag time, river flows, storage releases, hydrologic and withdrawal losses, and energy generation at hydropower plants, are input data for the HWBEG Model. They are used in conjunction with mathematically derived reservoir rules to simulate the river basin with all projects in place. Once a good correlation is obtained between actual reported data and the simulated energy generation with all projects on line, the regulating effect of each headwater storage project is removed from the model in order to determine the energy gains. The following flowchart outlines the procedures followed to calculate energy gains using the HWBEG Model.

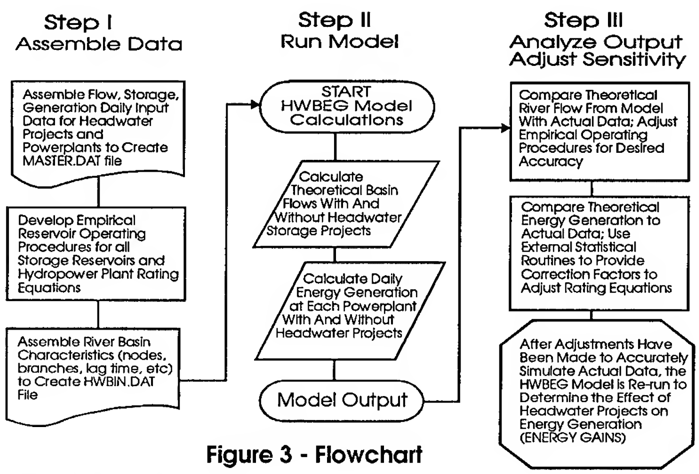

Figure 3 - Flowchart

PROGRAM MANAGEMENT

From an administrative perspective, managing the Commission's headwater benefits program is extremely challenging. In the early years of the program, a major obstacle in conducting investigations was the availability and assimilation of the necessary data required to render an assessment for benefits received. The use of the HWBEG computer model has standardized energy gain calculations and allowed more frequent investigations. In addition, the adoption of standard regulatory procedures for HWB determinations by the Commission was intended to achieve an expeditious assessment with fewer disputes over complex issues. These procedures, which were codified in 1986, have been tested by challenges to several HWB investigations. While the Commission's regulations provide an equitable procedure to determine headwater benefits, resolving individual assessments with beneficiaries can result in a protracted collection process. Even so, in administering its legislative responsibilities, the Commission has determined through HWB investigations, that federal projects provide significant energy gain benefits to downstream hydropower plants as shown below:

	Federal Projects Providing Energy Gains	Downstream Powerplants Receiving Energy Gains	Annual Energy Gains Provided/Received (Mwh)
Bureau of Reclamation	19		8,312,996
Corps of Engineers	98		5,669,898
Total	117	101	13,982,894

In 1978, Congress enacted comprehensive energy legislation that included regulatory relief for certain small hydropower projects, as well as other incentives for hydropower projects. As a result, many new hydropower plants have been constructed within the last decade (Figure 1). In addition to reviewing each project for its energy gain potential, the Commission must continually assess and update existing basin investigations to account for changes that would result in re-apportioned assessments. The Commission is assembling its list of river basins and establishing priorities for future headwater benefits investigations. Figure 3 shows the regional distribution of basins currently subject to HWB charges for federal projects as well as new basins under review.

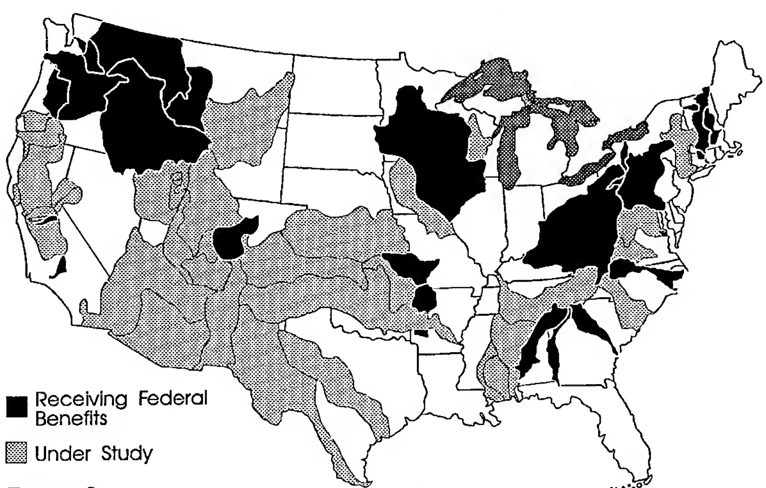

Figure 3
River Basins with Headwater Benefits

FUTURE PROGRAM ENHANCEMENTS

The Commission continues to strive to make headwater benefits an understandable issue. While the assessment and collection of charges for increased energy production are often a sensitive subject for most beneficiaries, it represents energy they would not otherwise have been able to produce. The assessments levied represent an equitable charge for what it would cost them to produce an equivalent amount of power.

The Commission recently completed the process of converting the HWBEG computer model to run on desktop computers. The model has

been converted to the FORTRAN language format, which will allow for wider distribution and familiarity with its use. In addition, the model includes several enhancements as follows:

1. The ability to analyze energy gains on a monthly basis. For river basins that are relatively stable, this could shorten yearly calculations by a factor of 30;

2. The ability to accurately model consumptive and evaporative losses from a river basin system;

3. The inclusion of turbine efficiency curves that will allow more flexibility in modeling existing hydropower plants; and

4. Increased flexibility in developing input data and formatting output results.

Future headwater benefits investigations should involve all parties from the beginning of a study in a cooperative effort. Although the parties may continue to disagree on the amounts to be assessed, every attempt is being made to eliminate the technical grounds for disputes. The current regulations provide a sound basis for achieving the Commission's goal of equitable, expeditious, efficient, accurate, and predictable HWB investigations.

REFERENCES

(1) 41 Stat. 1063. The Federal Water Power Act was amended by 49 Stat. 838 (1935), when it became the Federal Power Act.

(2) Federal Power Commission/Federal Energy Regulatory Commission, Annual Reports, 1934-1991.

(3) Code of Federal Regulations, Title 18, Conservation of Power and Water Resources, Section 11, Subpart B-Charges for Headwater Benefits, 1992.

(4) 82nd Congress, House Committee Print No. 23, The Allocation of Costs of Federal Water Resource Development Projects, December 5, 1952.

(5) Oak Ridge National Laboratory, The Headwater Benefits Energy Gains (HWBEG) Model Description and Users Manual, 1984.

THE RESULTS ARE IN
Auditing Hydropower License Compliance

Gail Ann Greely and Katherine E. Reed[1]

INTRODUCTION

"When the Federal Energy Regulatory Commission established its Division of Project Compliance and Administration and established rules on civil penalties for noncompliance, those who hadn't seen it coming finally realized that the "Age of Compliance" had arrived in the hydropower industry. If hydropower follows the pattern of other environmentally regulated industries, "environmental audits" will soon become a common tool for assuring compliance with the ever-greater levels of environmental regulation."

Those words were the beginning of a paper we presented at WaterPower '89. In the ensuing four years, compliance audits of hydroelectric facilities have begun--audits initiated by hydro licensees as well as audits initiated by the F.E.R.C. staff. Although the pace has been slower than we had anticipated, enough reviews have been performed to present an interesting comparison of the findings thus far.

A comparison of the audits conducted by our firm and those conducted by the F.E.R.C. staff within the Division of Project Compliance and Administration (DPCA) paints an interesting picture of the evolution of compliance auditing within the hydropower industry. We would like to acknowledge the assistance of Mr. Mark Robinson and Mr. Dean White with the F.E.R.C.'s Division of Project Compliance and Administration for providing data on the DPCA audits.

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[1] Gail Ann Greely, President, and Katherine E. Reed, Vice President, Pacific Hydro Consulting Group, Incorporated, 2150 Mariner Square Drive, Suite 101, Alameda, CA 94501

While this paper is limited to a discussion of environmental audits conducted at hydro projects, it should be noted that compliance with dam safety issues is also a major concern within the F.E.R.C. as well as the hydropower industry. The F.E.R.C.'s Division of Dam Safety and Inspections (D2SI) is responsible for overseeing compliance with the dam safety and public safety requirements of the Commission's regulations. D2SI monitors compliance through operations inspections conducted at regular intervals by the F.E.R.C. Regional Office staff, and through review of recommendations made by independent consultants retained by licensees to inspect F.E.R.C. licensed dams at five-year intervals. Dam safety violations continue to be the major focus (both in terms of number and dollar amount) of compliance orders and civil penalties issued by the F.E.R.C. to date.

DECIDING TO AUDIT

Our firm has conducted three independent compliance audits of federally licensed hydropower projects--one for a public utility, one for a water agency and one for an irrigation district. In each case our audit concentrated on one project, although the public utility operates several hydroelectric projects and has scheduled additional audits for later this year. Results of subsequent audits will be available for the oral presentation of this paper in August 1993.

In each of the compliance audits we have conducted, commissioning the independent audit was a reflection of the conscientious effort each licensee makes toward complying with the terms and conditions of its F.E.R.C. license. Compliance audits were used to affirm the licensee's ongoing commitment to project compliance. That's not to say that we didn't find a few minor infractions; however, in general the licensees' compliance records were admirable.

The audits initiated by DPCA paint a different picture. In determining which licensees to audit, DPCA initially reviews compliance records, looking for patterns of frequent problems. Those licensees found with a poor compliance history, ongoing problems or complicated compliance issues are targeted for compliance audits. While DPCA audits are usually directed toward one project, a licensee with numerous compliance problems may well be targeted for audits on additional projects. Unlike F.E.R.C. operations inspections that are held at specific intervals, compliance audits are usually one-time events aimed at evaluating specific compliance problems.

During fiscal year 1992 (October 1991 through September 1992) 36 audits were conducted by DPCA. During fiscal year 1993, DPCA has scheduled 24 audits, ten of which had been conducted as of April 1, 1993. A significant DPCA

finding of those projects audited thus far is that smaller projects are having disproportionately more compliance problems than larger projects. In many cases, continuing problems can be attributed to limited project revenues. Cash flow constraints reduce the number of options licensees feel they can afford to undertake to address a compliance issue.

AUDIT FORMAT

The compliance audits conducted by DPCA are limited to a review of the licensee's compliance with the terms and conditions of its F.E.R.C. license. The audits are usually conducted in a single two to three hour visit to the project site. A team of DPCA staff members representing disciplines in recreation and land management, fisheries and wildlife biologists, civil engineering, and hydro compliance visit the project site, focusing on those areas where compliance has been identified as a concern. The audit also provides DPCA with in formation on the licensee's specific day-to-day problems in trying to comply with the terms and conditions of the license.

Audits initiated by the licensee are usually much broader in scope. Not only do the audits address compliance with the F.E.R.C. license, but they are also tailored to provide information on project efficiency and measure performance against expectations. Agreements with resource and land management agencies are also reviewed to ensure that the licensee is satisfying its obligations under those agreements.

Audits conducted by our firm have identified areas of operation that, with minor revision, not only increased the likelihood of compliance, but could also lead to savings in time and money. Minor modifications in gaging equipment allowed one licensee to stop releasing minimum flows in excess of those required merely to ensure compliance. For another project, negotiations for a land exchange between the licensee and a federal land management agency were found unnecessary when a review of project exhibits identified the federal lands as project lands withdrawn under Section 24 of the Federal Power Act. The hydro potential of minimum flow releases from diversion structures were identified and the cost-effectiveness of developing those sites is now under investigation by another licensee.

COMPLIANCE PATTERNS

Patterns of audit findings also differed somewhat between those audits conducted by our firm and those conducted by the F.E.R.C. staff. Issues of minimum flow releases, gaging, run-of-river operation, recreation and other land management issues, general administrative requirements and the licensee's knowledge of F.E.R.C. regulations and procedures were most often found by

both DPCA staff and our firm as having compliance implications. While there do not appear to be many geographical patterns to compliance violations, seasonal patterns have developed. Most compliance activity occurs between the months of April and September, with the lowest allegations arising between November and January. The period of April through September not only coincides with the high recreation season, meaning more people in the project area to observe possible violations, it also corresponds to the period when most F.E.R.C. inspections and compliance audits occur. The period also coincides with low water periods that could impact minimum flow requirements.

Minimum Flows

Perhaps due to the high visibility minimum flow issues received in the west during the 1980's, minimum flows *per se* did not emerge as a significant compliance issue in the audits conducted by our firm. Licensees appeared to have a good handle on what their flow requirements were and had taken extra steps to ensure that those flows were met. However, the F.E.R.C. staff audits identified minimum flows as a major area of continuing noncompliance, especially in the eastern parts of the United States.

Gaging Requirements

Gaging issues were found as potential areas of noncompliance in audits conducted by our firm as well as those conducted by DPCA. Even when minimum flows were not regarded as a compliance issue, the location and rating of gaging stations were identified as potential areas of noncompliance in all three of the audits we conducted. In each instance the licensee believed that minimum flows were being recorded properly. However, in one instance we found that a conversion table used to calculated flows was inaccurate, and in two other instances we found that the gages used to document minimum flows were not situated at the locations specified in the F.E.R.C. license.

In every case, not only was the licensee unaware of the potential problem, but state and federal resource agencies consulted on gaging issues had not indicated a problem with current procedures, leading the licensees to a false sense of security. (See "Who's in Charge" below)

Run-of-River Operation

Gaging issues overlapped with concerns on run-of-river operations. Too often audits conducted by DPCA were the first time a licensee had been compelled to demonstrate that the project was being operated within the constraints of the F.E.R.C. license. Poor record keeping and inadequate gaging equipment contributed to instances of noncompliance with run-of-river operating criteria.

Unknowledgeable licensees were often unaware they were expected to "prove" that the project was being operated properly. So long as water's available, why can't we operate the project?

Recreation Facilities

DPCA audits frequently found unacceptable maintenance of recreation facilities. Often licensees contract for the construction, operation and maintenance of their recreation facilities, and fail to remember that they are held ultimately responsible by the F.E.R.C. license for proper operation and maintenance of the facilities. Vandalism aside, proper facility maintenance is an ongoing responsibility imposed by the F.E.R.C. license.

Design, installation and location of project recreation signs, identifying the project as an F.E.R.C. licensed facility, also presented opportunities of noncompliance. While some licensees may be inclined to gloss over the Part 8 requirements, the F.E.R.C. staff takes these regulations as seriously as other provisions in Title 18 of the Code of Federal Regulations.

The recreation facilities we inspected were all found well above-average in their maintenance, as well as design of the facilities provided. The compliance issue we encountered most often was the unauthorized construction and/or expansion of recreation facilities contrary to an approved recreation plan or totally absent any F.E.R.C. notification. In their desire to provide adequate recreational opportunities at project facilities, we have found it not uncommon for a licensee to design and construct a facility without seeking F.E.R.C. review and/or approval.

Land Management

General land management issues that raised the potential for non-compliance included the inconsistent administration of the land use article included within many F.E.R.C. licenses. We found confusion over exactly which land issues require prior F.E.R.C. approval. Similarly, DPCA has issued a form letter to those licensees reporting conveyances of project lands for non-project uses. The letter reminds licensees that filing an annual report of conveyances is not the culmination of their responsibilities. Licensees have an ongoing responsibility to ensure that the non-project use of the land continues to meet all applicable environmental standards.

Administrative Requirements

Inadequate record-keeping and reporting procedures ranked high on the list of compliance issues identified during DPCA audits. Documenting minimum

flow requirements, maintaining adequate project records, and making timely filings with the F.E.R.C. were often identified as areas of noncompliance during DPCA audits.

While some licensees feel they cannot afford sophisticated gaging equipment to continuously monitor minimum flows, that does not relieve them of the requirement to maintain adequate records to demonstrate minimum flow compliance. "Visual observations" are not sufficient to allow DPCA to confirm adequate compliance.

Failure to maintain project records can also result in instances of noncompliance. One requirement that is frequently overlooked, or often inadequately addressed, is the Commission's requirement to provide a copy of an approved recreation plan and the license instrument, properly indexed, for public inspection. We have seen many licensees make a copy of the recreation plan and original license and place them in the local office, never to be thought of again. The 18 CFR §8.2(b) requirement requires that the entire *license instrument* be made available. The license instrument includes not only the original license, but copies of all subsequent Commission orders that modify the original license and/or incorporate portions of approved plans into the license. For easy reference the documents must be indexed by subject matter and/or cross-referenced to subsequent orders.

Maintaining up-to-date lists of approved exhibit drawings is another requirement that is often overlooked. As project drawings are revised and/or superseded, a listing of current drawing numbers becomes more important. Another provision often overlooked is the requirement to keep original drawings (those previously approved by the Commission) for the full record-retention period prescribed in 18 CFR §125.2, even though the original drawing has been superseded.

Late submittal of required filings often triggers a compliance audit by the DPCA. If a licensee has difficulty with the paperwork requirements, what else may be going on at the project that deserves review? We are always surprised to talk with licensees who really don't have a clue what their license requires of them. Or, if they have a good idea what the license requires, they don't know that there are numerous 18 CFR requirements that supplement their license. Conscientious monitoring of required filing dates, whether annual submittals or one-time occurrences, is a first step toward adequate license compliance.

Who's in Charge?

According to DPCA, many compliance problems are caused through the licensee's misunderstanding of "who's in charge." Licensees are faced with

satisfying many masters and must have a clear understanding of which requirements take precedence. Minimum flow requirements, including dry year criteria and timing of flows, are often found in various resource agency agreements as well as the F.E.R.C. license. Similarly, recreation facility design, operation and maintenance may be covered in a federal land management agency special use permit or memorandum of understanding. Licensees have been surprised to learn through an audit that, although they have been operating their project with the complete support of state and federal agencies, they have in fact been violating the terms of their F.E.R.C. license. An audit can identify overlapping obligations and clarify a licensees understanding of when F.E.R.C. in is charge and when other agency requirements take priority.

Audit Follow-up

Following an audit conducted by DPCA, noncompliance items identified during the audit are discussed with the licensee on-site and then treated like any other noncompliance issue. The matter is assigned to an appropriate DPCA staff member for additional follow-up with the licensee in the form of letters and, if necessary, a compliance order. DPCA encourages the licensee to respond with specific actions to avoid future violations. Depending on the severity of the identified violation and the licensee's willingness to rectify the problem, the violation may or may not result in proposed civil penalties.

Following an audit initiated by the licensee, the issue then becomes what to do with the additional information obtained from the audit. A thorough audit will identify where violations have occurred or are likely to occur as the result of inadequate safeguards. It then becomes incumbent on the licensee to implement audit recommendations and to notify the F.E.R.C. regional office and/or DPCA with proposed measures to correct the current violation and to avoid future violations.

An audit can be a positive vehicle for addressing compliance violations. If violations are discovered and the licensee reports audit results to F.E.R.C. staff, the licensee's actions can be a mitigating factor in determining the consequences of noncompliance. On the other hand, if an audit identifies compliance problems and the follow-up is inadequate to ensure future compliance, the opposite could result. Failure to correct known compliance violations will place the licensee in greater jeopardy of civil penalties.

Audits conducted by our firm have included such follow-up recommendations as seeking modification or elimination of F.E.R.C. license articles, notifying F.E.R.C. of modifications made to project facilities, and installing additional gaging equipment.

One example of a successful exchange between a licensee and F.E.R.C. regional staff occurred as a result of an internal audit conducted by a client that identified minimum flow violations over a previous 18-month period. While the violations had not been identified during the F.E.R.C. operations inspections, once known, the licensee approached the F.E.R.C. Regional Office staff with the information they had gathered and proposed methods to ensure that the error would not be repeated in the future. While the Regional Office staff felt obligated to report the compliance violations to DPCA, their report praised the licensee's candidness and direct approach to resolving the problem. DPCA noted the noncompliance violations, but did not pursue further action against the licensee.

Selecting the best approach to respond to audit findings must take many factors into consideration: severity of the noncompliance, its causes, available remedies and many others.

Audit Recommendations

When asked to provide general recommendations to licensees that may be faced with an audit, DPCA staff offered the following suggestions.

Review -- the license or exemption articles
terms and conditions of the license or exemption
correspondence with the F.E.R.C., resource agencies and general public
amendment orders
orders approving plans

Gather -- operating records
minimum flow records
instrumentation data
recreation data

Contact -- state and federal resource agency local offices to see if they have any problems with project operations

Think -- of questions **you** want to ask **F.E.R.C.**

Unlike an internal audit that allows licensees to "gear-up" for the audit, licensees may not be given advance notice of a DPCA audit. Even in those cases where an audit is pre-arranged, the notification time is usually at most a few days. The more licensees can do to be prepared for an audit, the more successful the results are likely to be.

Not surprisingly, many of the audit recommendations made by our firm mirror those made by DPCA staff. Recommendations similar to those listed here were included in each audit report and are appropriate for use by licensees planning to audit their own hydro facilities.

1. Prepare and maintain a License Binder containing a complete copy of the existing license, with appropriate revisions to license articles, approved plans, and other documents incorporated by reference.

2. Establish procedures to review ongoing compliance obligations, at least annually, and provide reminders for preventive measures.

3. Establish procedures to respond to incoming requests, complaints and compliance obligations in the event primary project personnel are unavailable.

4. Establish procedures to review contents of project files in the F.E.R.C. Regional Office and Washington, D.C., at least annually.

5. Educate staff of the types and magnitude of changes to project facilities and operations requiring prior F.E.R.C. approval.

6. Request the F.E.R.C. to issue errata to correct inaccuracies in the license and to revise and/or remove license articles that contain inaccuracies or are not longer required.

7. Establish procedures to follow up on F.E.R.C. review of any filed complaints, investigations, and/or filings to ensure that closure is obtained.

8. Prepare a list of approved and superseded exhibit drawings to ensure that a complete set of drawings is available.

9. Review approved exhibit drawings to ensure that approved project boundary is consistent with current use and project needs and revise exhibits accordingly.

10. Review F.E.R.C. record retention requirements to verify that licensee's policies will ensure continuing compliance.

11. Routinely request copies of all inspection reports (Operations, Environmental and Public Use, and any special reports) prepared by the F.E.R.C. staff.

12. Review F.E.R.C. inspection reports to determine whether additional actions may be appropriate to ensure ongoing compliance.

13. Periodically review agreements with recreation concessionaires to ensure consistency with approved recreation plan and F.E.R.C. regulations.

14. Monitor effectiveness of public safety devices and notify F.E.R.C. of removal of any existing facilities or installation of new devices.

15. Prepare an inventory of project signs, including location and content of signs, and conduct regular inspections of project signs.

Future Projections

Since 1987 we have written numerous papers and articles for hydropower conferences and publications predicting increased F.E.R.C. emphasis on enforcing compliance by licensees with the obligations of their licensees. We have made numerous suggestions based on our experience to assist licensees in facing the challenges of compliance issues. It is a message that too many licensees don't want to hear. Wake up out there! Compliance management must be a priority!

Negotiating the Maze: Hydroelectric Development and Relicensing on Federal Lands

Michael A. Swiger
Daniel J. Whittle[1/]

I. ABSTRACT

This paper focuses on the overlapping and potentially conflicting federal regulations facing licensing and relicensing applicants for hydroelectric projects on federal lands. A better understanding of the current network of federal regulation, together with careful planning, may allow applicants to anticipate potential conflicts in advance, thereby resulting in a more expeditious and cost-effective licensing process.

II. INTRODUCTION

The past several years chronicle a growing fragmentation of the hydroelectric licensing authority of the Federal Energy Regulatory Commission (FERC). This trend particularly has affected development of water power on federal lands. It also has serious implications for the approximately 300 projects which will undergo FERC relicensing in the next several years.

The U.S. Department of Agriculture Forest Service (Forest Service) and the U.S. Department of Interior Bureau of Land Management (BLM), rely increasingly upon their respective land and resource management planning authorities to influence licensing and relicensing decisions. Both agencies have asserted authority under the Federal Land Policy and Management Act (FLPMA) to require FERC license applicants to obtain special use authorizations (SUAs), or rights-of-way, in addition to obtaining licenses from

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1/ Mr. Swiger is a Member, and Mr. Whittle is an Associate, of the law firm of Van Ness, Feldman & Curtis, A Professional Corporation, 1050 Thomas Jefferson Street, N.W., Washington, D.C. 20007.

FERC. Federal agencies also have more aggressively utilized their authority under § 4(e) of the Federal Power Act (FPA) to impose mandatory license conditions on projects located on federal reservations. Another favored strategy is to invoke the National Wild and Scenic Rivers Act (WSRA) to delay or block FERC from licensing projects on river segments within the agency's land management jurisdiction.

As a result, project developers and owners face additional layers of costly, time-consuming, and often duplicative permitting requirements. The discussion below provides an overview of the statutory framework underlying the regulation of hydro development on federal lands, and discusses various approaches taken by federal agencies to assert control over the licensing and relicensing of projects. These include both projects on public lands (i.e., federal lands subject to private appropriation and disposal under the public land laws) and federal reservations (i.e., lands not subject to such appropriation or disposal). An introduction to the network of federal permitting requirements will assist project owners and developers in avoiding and/or planning for potential problems, delays, and conflicts.

III. THE STATUTORY MAZE

A. The Federal Power Act

The Federal Water Power Act of 1920 (now the FPA) centralized federal regulatory authority over non-federal hydroelectric development in a single agency, the Federal Power Commission (now FERC). Congress' purpose was to eliminate the earlier scheme of duplicative and conflicting federal regulation, and to promote comprehensive and rational development of the nation's water resources in a way which balanced power and non-power values. The U.S. Supreme Court has held that the FERC's jurisdiction over hydroelectric projects is exclusive absent express modification by federal legislation.

The FPA itself contains some limited exceptions to FERC's exclusive authority. For example, the U.S. Army Corps of Engineers (Corps) must approve plans for structures affecting navigable capacity. Section 18 of the FPA provides that the Secretary of Interior or the Secretary of Commerce may prescribe fishway conditions which must be included in the project license. For projects within national forests, Indian reservations and other federal reservations, § 4(e) of the FPA provides that licenses for such projects "shall be subject to and contain such conditions as the Secretary of the department under whose supervision such reservation falls shall deem necessary for the adequate protection and utilization of such reservations."

Finally, the FPA prohibits the Commission from licensing projects within national parks or national monuments without specific authorization from

Congress. This prohibition does not extend to existing projects located on private lands within park unit boundaries, nor to projects that were constructed prior to park designation. The National Energy Policy Act of 1992 extended the prohibition to new hydro projects proposed to be located on non-federal lands within the boundaries of any unit of the National Park system that would have a direct adverse effect on Federal lands within such unit.

B. Section 501 of FLPMA -- Rights-of-Way over Public Lands and National Forests

Section 501 of FLPMA authorizes the Secretary of Agriculture, with respect to national forest lands, and the Secretary of Interior, with respect to public lands, to grant rights-of-way over such lands for a variety of purposes, including for "systems for generation, transmission, and distribution of electric energy . . . ." Subsequently, the BLM and Forest Service began requiring FERC license applicants to obtain SUAs.

FERC initially resisted efforts by the agencies to extend FLPMA § 501 requirements to FERC license applicants. In the Henwood case, FERC reversed its long-held position and ruled that FLPMA permits were required. It was then overturned by the Ninth Circuit Court of Appeals. In holding that FLPMA rights-of-way are not required for FERC licensed projects, the Court explained that it would not "lightly imply a repeal of the exclusive nature of FERC's authority over the conditions of use of federal lands for hydropower projects."

In the National Energy Policy Act of 1992, Congress overturned Henwood by modifying § 501 of FLPMA expressly to extend rights-of-way authority to the Secretaries of Interior and Agriculture over FERC projects on public lands and national forests. However, it included a "grandfather" clause for existing projects which (1) have never held FLPMA permits, and (2) are not planning to expand operations onto additional national forest or public lands. This effectively exempted from FLPMA most, if not all, of those projects in the current wave of relicensings. As discussed below, it left unresolved whether the Forest Service could rely on FPA § 4(e) to bootstrap in an SUA requirement.

C. Land and Resource Management for the National Forests

Forest Service administration of the national forests is governed primarily by three federal statutes. First, the Forest Service Organic Act of 1897 (Organic Act) provided two specific purposes for which public lands could be reserved for national forests: (1) to secure favorable conditions of water flows, and (2) to furnish a continuous supply of timber for the use and necessities of citizens of the United States. Second, the Multiple-Use Sustained Yield Act of 1960 (MUSYA) expanded the management authority of the Forest Service to include several additional purposes -- outdoor recreation, range, timber, watershed, and

wildlife and fish. Third, the National Forest Management Act of 1976 (NFMA) sets forth a procedural framework for forest planning and management. NFMA directed the Forest Service to prepare long-term land and resource management plans (forest plans) for each national forest by 1985, and established a series of procedural and substantive requirements for developing, implementing, and revising such plans. Under NFMA, forest plans are generally programmatic in nature, meaning that they identify broad goals and objectives, establish management standards and guidelines, and set management prescriptions (i.e., scheduled combination of uses, outputs and activities allowed for any given management area). Site specific decisions are made pursuant to these standards, guidelines, and prescriptions. It is also through forest planning that the agency usually determines which rivers (or segments thereof) are eligible for designation under the WSRA.

Once finalized and approved by the regional forester, a forest plan remains the primary guidance document for forest management until it is amended or revised according to the terms and procedures set forth in NFMA and the implementing regulations. NFMA provides that if any proposed plan amendment would constitute a "significant change," then the Forest Service must provide opportunities for public participation similar to those provided for plan development (see below Section V). The agency must revise each plan as a whole every 15 years, or earlier if forest conditions have "significantly changed."

It is against the forest plan's standards and guidelines that the Forest Service determines and justifies the conditions imposed on hydro development under § 4(e) and FLPMA § 501. Thus, if the national forest is in the process of developing or revising a particular forest plan, the Forest Service may ask FERC to delay issuance of the license until such plan is final.

D. Land and Resource Management of the Public Lands

Unlike the Forest Service, the BLM lacks authority under § 4(e) to impose mandatory conditions on FERC licenses. It does have the same right-of-way authority, however, as does the Forest Service under FLPMA § 501. FERC also has discretion under the FPA to incorporate BLM recommendations into the license. Thus it is important, for applicants to become familiar with BLM's land use planning process.

FLPMA directed the BLM to conduct a system-wide, comprehensive inventory of federal public lands and then to "develop, maintain, and, when appropriate, revise land use plans" to guide the use of federal public lands. As is the case with forest plans, BLM land use plans, or so-called resource management plans (RMPs), establish management objectives and areas, designate allowable uses for particular management units, and set target levels of resource production. RMPs must be based upon the "principles of multiple

use and sustained yield," be developed through a "systematic interdisciplinary approach to achieve integrated consideration of physical, biological, economic, and other sciences," and "give priority to the designation and protection of areas of critical environmental concern." Like forest plans, RMPs must be developed in accordance with the National Environmental Policy Act (NEPA).

FLPMA requires the BLM to solicit public involvement in the development of RMPs. Under FLPMA regulations, the public may participate early in the planning process to identify issues that should be addressed in the plan. Since development of the NEPA environmental impact statement (EIS) for the plan generally proceeds contemporaneously with plan development, this early public participation also usually serves as the scoping process for the EIS.

E. Wild and Scenic Rivers

Section 7 of the WSRA prohibits FERC from licensing any project "on or directly affecting" any river that has been designated by Congress for inclusion or for potential inclusion in the wild and scenic river system. WSRA also directs federal agencies through their planning to identify rivers that have potential for future designation. The Department of Interior maintains a nationwide rivers inventory (NRI) of those rivers for possible addition to the system.

For national forests, it is necessary to look to the particular forest plan in order to identify whether a WSRA conflict may exist. In developing forest plans, the Forest Service examines all rivers designated by Congress for study, rivers included in the NRI, and other rivers having potential as a wild, scenic or recreational river. Typically, the agency engages in a three-step process. First, upon determining that a particular river might be appropriate for inclusion in the WSRA system, the Forest Service identifies such river as "eligible" in its forest plan. Second, the agency determines the appropriate classification (wild, scenic or recreational). Third, the agency conducts a suitability study. Pending a determination of suitability, agency policy is to protect those eligible segments from any activities that might disturb or diminish the qualities that make them eligible for designation. These qualities include free flowing characteristics, water quality, scenic, recreational, cultural, wildlife and fisheries. Thus, for any proposed hydro project located "on or directly affecting" any river segment designated as eligible under the forest plan, the Forest Service recommends to FERC that no license be issued until a suitability study on that particular segment is complete.

FERC's current policy is to honor such Forest Service recommendations by either (1) waiting until a suitability study is complete before making a final determination as to whether license should issue, or (2) denying the license application without prejudice. In the latter case, if the suitability study releases the segment for further development, or if Congress denies a WSRA designation,

FERC will reconsider the application. FERC policy here only applies to licenses, not preliminary permits. FERC generally issues preliminary permits for projects on designated segments, unless it is clear that a project cannot be reconfigured to avoid violation of the Act.

Designation of a river segment for inclusion in the wild and scenic river system any time before the license becomes final can kill a proposed project. For example, in the 1992 Webster case, FERC ruled that where a license is issued but not final (e.g., meaning where the time for judicial review has not lapsed) Congress can designate a river segment as wild, scenic or recreational and take the license. If the license is final when such designation is made, however, then the licensee is entitled to receive due compensation.

IV. FPA § 4(e) CONDITIONING AUTHORITY -- HOW FAR DOES IT GO?

A. Scope of Conditioning Requirements

For new or expanding projects in national forests, the scope of § 4(e) conditioning authority is no longer of practical significance. Under the recent amendment to FLPMA, the Forest Service has virtual veto authority over new hydro development in national forests. The scope of § 4(e) conditioning authority is still an important and unresolved issue, however, for new and existing projects located on Indian reservations and military reservations, as well as for relicensing of existing projects located within national forests.

The U.S. Supreme Court in the famous Escondido case concluded that § 4(e) requires FERC to accept, without modification, any conditions recommended by that federal agency having jurisdiction over a particular federal reservation. In doing so, however, the Court ruled that there are "limits on the types of conditions" that the Secretaries can impose on licensees. The most important of these is that "the Secretary has no power to veto the Commission's decision to issue a license and hence the conditions . . . must be reasonably related to the protection of the reservation and its people." However, the Court held that only federal courts, not FERC, have the authority to reject conditions imposed as exceeding the permissible limits. Thus, even when FERC considers § 4(e) conditions imposed by the land management agency to be unreasonable, it has no authority to modify such conditions. FERC's only options appear to be 1) to issue the license with § 4(e) conditions attached, or 2) refuse to issue the license as conditioned. If FERC chooses the first option, it could issue the license with objections, thus adding to the record if the licensee opts to challenge the conditions in the federal court of appeals. Escondido also arguably leaves room for FERC to make a threshold decision on whether conditions are in fact § 4(e) conditions at all, and to reject any of those it determines are not legitimate § 4(e) conditions.

Escondido left open the question of whether an agency can impose § 4(e) conditions on project relicensing. Nonetheless, in a complete reversal of its long standing policy on the issue, FERC held in its 1989 Pasadena decision that, because § 4(e) is not expressly limited to original licenses, § 4(e) does in fact apply to project relicensing. As Commissioner Trabandt pointed out in dissent, this abrupt policy reversal contradicts the plain language as well as the legislative history of the FPA. Commissioner Trabandt argued that § 4(e) applies only to initial licenses, while FPA § 15 is the exclusive authority governing the issuance of new licenses. Section 15 does not grant similar mandatory conditioning authority to federal agencies. Trabandt cautioned that "the effect of applying the mandatory conditioning authority of § 4(e) is to severely weaken, if not destroy, the Commission's capability to protect [the interests of investors and the project's customers] as Congress intended."

In the aftermath of Escondido, FERC has shown increasing reluctance to protest or even question an agency's § 4(e) conditioning authority. This reluctance is well illustrated by FERC's routine acceptance of the standard § 4(e) conditions currently imposed by the Forest Service. These require, among other things, that applicants for both original and new licenses request a SUA from the agency. For example, in its September 1992 order issuing new license for Pacific Gas & Electric's ("PG&E's") Phoenix Project, FERC included the standard § 4(e) license condition from the Forest Service requiring the licensee to obtain a SUA within six months of the issuance of the license and before commencing any "land disturbing" activities. FERC's order made no reference to Henwood, which had held that FLPMA does not apply to FERC licensed projects.

FERC's continued acceptance of this standard SUA condition for relicensing goes beyond the rule of Escondido and squarely contradicts Henwood. As PG&E pointed out in its request for rehearing to FERC, Escondido only prohibits FERC from second guessing the "reasonableness" of Forest Service § 4(e) conditions, but does not prevent FERC from rejecting those conditions that are clearly outside the scope of § 4(e). The SUA condition seems to clearly fall outside the scope of § 4(e) because, for all intents and purposes, it constitutes a separate and distinct licensing process for FERC license applicants rather than a valid § 4(e) condition related to "adequate protection and utilization" of the national forest. In the Phoenix case, the Forest Service's draft SUA included the following requirements: 1) licensee would be subject to all regulations of the U.S. Department of Agriculture, including those related to the protection and enhancement of fish and wildlife; 2) licensee would be required to protect scenic and aesthetic values; and 3) licensee would be required to develop a recreation plan.

In addition, since the Forest Service does not typically issue SUAs until six months or more after the issuance of the FERC license, FERC is unable to

help create a record for appeal if it does not even know what the conditions will be. Applicants are faced with the same problem.

B. Finding of Interference and Inconsistency

Section 4(e) also conditions the issuance of a hydro license on a finding by FERC that such project will not interfere with, or be inconsistent with "the purpose for which such reservation was created or acquired." FERC has for all intents and purposes abdicated its authority to make such a finding to the Forest Service. Where the Forest Service determines in a forest plan that hydro development is inconsistent with other uses and/or management objectives of that particular national forest, FERC's present policy is to defer to such determination.

This policy is illustrated in its October 1992 Nelson order denying applications for license for three proposed hydro projects in the Boise National Forest in Idaho. There FERC's final EIS for the proposed projects concluded that sedimentation and turbidity would increase during project construction, but that if the construction of the three projects were staggered over a period of time, construction-related water quality impacts would be negligible. The EIS recommended licensing of the projects with certain mitigative conditions. However, the final forest plan for the Boise National Forest subsequently concluded that construction-related water quality impacts rendered the projects incompatible with fishery enhancement objectives. Based on this finding, the Forest Service concluded that construction of the proposed projects would "interfere with the purposes for which the Boise National Forest was created or acquired," and recommended that FERC deny the license applications. FERC then reversed its earlier finding of consistency and denied all three applications.

Commissioner Trabandt in dissent blasted the Commission for abdicating its § 4(e) responsibility to the Forest Service. He noted that Congress establishes the purposes for which national forests are "created or acquired," and that the Forest Service lacks the authority to determine such purposes each time it revises a forest plan. This criticism is supported by the U.S. Supreme Court's decision in United States v. New Mexico, a case involving federal reserved water rights. There the Court held that the MUSYA broadened the express purposes for which the national forests were to be managed prospectively, but did not retroactively modify the narrower purposes for which national forests had been created or acquired before 1960.

Since the amendment to § 501 of FLPMA in December 1992, the Forest Service has authority to grant or deny rights-of-way to new or expanding hydro projects. If the agency determines that hydro development is incompatible with the Forest plan it can simply deny the SUA, thereby eliminating the need for a consistency finding by FERC under § 4(e). What is not yet clear, however, is

whether FERC will defer to the Forest Service in making § 4(e) consistency determinations with respect to relicensing of existing projects. The Forest Service looks to the specific management objectives contained in particular forest plans when making a recommendation on whether hydro development is consistent with the purpose for which the national forest was "created or acquired." Given the fact that these objectives are subject to change during each revision of a forest plan, project owners face the threat that the Forest Service would determine that even an existing project is no longer consistent with the purposes for which the forest was "created or acquired."

Until § 4(e) issues are resolved by the courts, or Congress, projects up for relicensing are in a dilemma. The Forest Service does not usually determine § 4(e) conditions until late in the licensing process, at which point licensees will already have expended substantial time and money. If FERC continues to be unwilling to question the Forest Service's assertion of § 4(e) authority over existing projects, project owners will only be able to seek relief either in the courts or through the Forest Service itself.

V. FOREST PLANNING PROCESS

Because forest plans and RMPs might be used either to kill hydro projects in national forests and public lands, or at a minimum, to support the imposition of additional FERC license conditions, project owners and developers should become familiar with and participate in the planning processes of the Forest Service and BLM in order to minimize licensing problems or conflicts. The discussion below outlines opportunities for public involvement in forest planning. Similar opportunities exist with regard to BLM's development of RMPs.

NFMA directs that forest plans be developed and revised in accordance with procedures under NEPA. Thus, project owners (as members of the public) have the opportunity to review and comment upon the draft EIS prepared in conjunction with the plan. Also, § 6(d) of NFMA provides additional opportunities for the public to participate in the "development, review and revision" of the plans. Typically, the Forest Service will solicit the involvement of the public early in the scoping process, in the review and comment of draft planning documents, public meetings, and occasionally through citizen working groups. Though NFMA regulations require the agency to be responsive to public input, the agency retains relatively broad discretion in establishing standards, priorities and management prescriptions in the final plan.

The public may protest the final forest plan through the agency's administrative appeals process, a relatively informal, non-adjudicatory process. Appeal of a forest plan can be made simply by submitting a letter of protest to the Chief of the Forest Service, subject to additional, discretionary review by the Secretary of Agriculture. These so-called "postage stamp appeals" may also be

made with respect to any NEPA-based decisions on specific projects or activities that flow from a plan. Participation in the planning process is not a prerequisite for appealing the final forest plan itself. However, for appeals of proposed actions under the plan, only those parties that participated in the comment period have standing to protest the proposed action.

Though NFMA does not specifically provide for judicial review, following a final decision on an administrative appeal any appellant can seek judicial review in federal district court under the Administrative Procedures Act (APA). As explained in the next section, however, FERC license applicants may appeal agency conditions imposed on FERC licenses directly to federal court of appeals under the FPA.

VI. CHALLENGING FPA § 4(e) CONDITIONS AND FLPMA § 501 REQUIREMENTS

If negotiations and administrative appeals with agencies fail, FERC license applicants may have to litigate. With respect to appeals of conditions imposed under § 4(e) of the FPA, the Ninth Circuit Court of Appeals in the Yeutter case made clear that § 313(b) of the FPA applies. Section 313(b) vests the federal courts of appeal with exclusive jurisdiction to review FERC license orders. In Yeutter, plaintiffs had brought an action in federal district court challenging the issuance of a FERC license, alleging that the § 4(e) conditions imposed by the Forest Service on the licensee violated both NEPA and the American Indian Religious Freedom Act. In reaching its conclusion, the court stated "the § 4(e) conditions imposed by the Service have no significance outside of the licensing process," and thus that the court of appeals was the only forum in which review could be sought.

It is not entirely clear, however, whether § 313(b) of the FPA applies with respect to the challenge of FLPMA § 501 requirements. While the same principles are present here as in the Yeutter case, (i.e., the FLPMA SUA has no significance outside of the FERC licensing process), statutory authority to issue the SUA lies outside of the FPA entirely. Thus, it is conceivable that courts would view an appeal of a FLPMA SUA in much the same manner as they view appeals of state water quality certifications under § 401 of the Clean Water Act. Courts have ruled that judicial review of § 401 certifications is controlled by state law, not FPA § 313(b).

Until this jurisdictional issue is resolved, FERC applicants may be forced to litigate FLPMA SUA requirements both through the FPA's judicial review procedures and through the district courts.

DEVELOPMENT OPTIONS AND OPPORTUNITIES

Peter C. Kissel 1/

Abstract

In the 1990's, a wide range of investor and other ownership interests is likely to be involved in hydroelectric project development in the United States. In the course of a few short years, rapid changes in economic, environmental, legal, financial, and political climates have dramatically altered traditional approaches and opened up new opportunities. Traditionally, the same entity (or small group) conceived and planned a project, obtained necessary government approvals in its own name, financed the project from a combination of internal funds and direct credit sources, and utilized the project output. For a variety of reasons, this traditional prototype is no longer typical. Now, a myriad of different entities, with different objectives, participating at different stages, is likely to be necessary for successful project development.

The variety of joint arrangements is virtually unlimited, and in many cases would have been unthinkable only a few years ago. Municipalities have undertaken joint ventures with investor owned utilities, independent developers have joined with municipals, banks have exercised interests in projects they have financed, operators are acting as investor-owners. Individuals, private equity funds, and limited partnerships are also all involved in some form of ownership. Many of these entities often participate in the same project at some stage or another, as projects experience a constant evolution of ownership, financing, and operation.

1/ Principal, Baller Hammett, P.C., Suite 1200, 1225 Eye Street, N.W., Washington, DC 20015.

These arrangements, of course, are not accidental. Rather, they are the conscious result of careful business planning which includes constantly monitoring the regulatory, economic, and political winds. Successful development requires its proponents to be creative, flexible, and responsive to changing circumstances. Any project developer -- whether investor owned utility, municipal body, individual, or independent developer -- must be prepared to expand, contract, or even relinquish, its interests as circumstances change.

This paper will examine some recent examples of the extent to which project development options have evolved over the course of several different project's lives, and discuss the roles and goals of the different types of entities mentioned above. The purpose of this paper is to provide developers with an appreciation of the different perspectives of the various entities likely to be involved, and to review the structural options available, and resulting opportunities, to bring projects on line and keep them running.

<u>Introduction</u>

On March 20, 1985, the Federal Energy Regulatory Commission ("FERC") issued Order No. 413, <u>2/</u> thereby instituting a sea change in the rules by which applicants for hydroelectric project licenses were required to address environmental issues. It soon became apparent that Order No. 413 was only one in a series of legal, financial, pragmatic and political changes about to dramatically affect the process by which project developers and financial investors could acquire interests in hydroelectric projects.

Order No. 413 was followed, in rapid order, by Congressional enactment of the Electric Consumers Protection Act of 1986 ("ECPA"), which adopted the first wholesale amendments to the environmental provisions of the Federal Power Act ("FPA") since 1935; enactment of the Tax Reform Act of 1986, which eliminated the 10% business tax credit; and expiration of the 11% energy tax credit for projects which were not licensed by December 31, 1988. Overarching all of these events was an evolving series of decisions by the FERC which

<u>2/</u> "Final Rule", <u>Application for License, Permit, or Exemption from Licensing for Water Power Projects</u>, Docket No. RM83-56, *FERC Statutes and Regulations* (CCH) ¶ 30,632 (1985); 50 <u>Fed</u> <u>Reg</u>. 11658 (1985).

significantly liberalized the policy by which financial interests could participate in projects without becoming co-licensees; and a growing public concern for the extent to which local governmental bodies should be involved in project development.

Principal Events Affecting Project Ownership

A brief summary of the principal features of each of these occurrences will assist in understanding the FERC cases of multifaceted project interests examined below. First, the FERC's Order No. 413 established the now famous (or infamous) three-stage agency consultation process [Congress is often improperly given the "credit" by environmentalists and others for forcing greater consideration of environmental issues in the licensing process; in fact, FERC was a year and a half ahead of Capitol Hill on this issue.] Order No. 413 took effect on June 10, 1985. From that date forward, any prospective license applicant was required to *first*, provide an initial consultation package to natural resource agencies, *second*, circulate a draft application to agencies for comments and reflect any comments received in the application, and *third*, invite additional agency comments after the filing of the license application. 3/

Second, ECPA amended the FPA by imposing additional environmental considerations on the licensing process. In essence, ECPA required the Commission to give "equal consideration" to the protection, mitigation of damage to, and enhancement, of fish and wildlife and other environmental resources, 4/ and required FERC to adopt the recommendations of fish and wildlife agencies made pursuant to the Fish and Wildlife Coordination Act, unless such recommendations are deemed by FERC to be inconsistent with the purposes of the FPA. 5/ This is the statutory provision which established the so-called "10(j) recommendations", named after the new section of the FPA in which it appears.

3/ The regulations enacted in Order No. 413 are codified in Section 4.38 of the Commission's Regulations Under the Federal Power Act, 18 C.F.R. § 4.38.

4/ 16 U.S.C. § 797(e).

5/ 16 U.S.C. § 803(j).

Third, the Tax Reform Act of 1986 repealed the investment tax credit of 10% for equipment placed in service after 1986. Although certain exceptions applied for projects subject to contracts already in place, this provision eliminated tax credits for business investments, including those in hydro projects.

Fourth, the 11% energy tax credit expired on Dec. 31, 1988 for projects which were not licensed by that date. Enacted as part of the Windfall Profits Tax Act of 1980, this provision had provided important incentive to the development of projects by small independent developers in the early 1980's. Originally set to expire in 1986, it had already been extended for two years, but was allowed to expire at the end of 1988, despite efforts to extend it further.

Fifth, the FERC over the past ten years has exhibited an ever-increasing receptiveness to innovative financing arrangements without requiring co-licensee status by every entity with a financial or tax ownership interest. Beginning with the Little Falls decisions 6/, FERC has adopted an evolving policy which accommodates investment interests without mandating a corresponding licensee status.

Sixth, the public has became more aware of the involvement of local governments in projects, and has demanded greater accountability. In some instances, heightened public awareness has been manifested by an increased public acceptance and even encouragement of municipal ownership; in other situations, this has led to devolvement of municipal participation in projects when appropriate.

The overall effect of these changes has been, on the one hand, to make the licensing *process* more time-consuming and expensive, and to remove certain tax incentives which allowed small entities and individuals to develop projects with a minimum of external financial assistance. On the other hand, these events correspondingly have prompted many developers to turn to new sources of financing, and have encouraged a greater participation by a wide range of interests. The first four events described above served to compound the

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6/ Little Falls Hydroelectric Associates, a Limited Partnership, 28 FERC ¶ 61,214 (1984); Little Falls Hydroelectric Associates, a Limited Partnership, 29 FERC ¶ 61,001 (1984).

licensing process, requiring a greater expenditure of resources and increasing the risk that a given project may not be developed due to financing or environmental considerations. This in turn opened up a need for developers, especially small independents with limited resources and municipalities which are directly accountable to public mandates, to turn to various outside entities to assist in the development process. The last two events mentioned above -- particularly FERC's broadening policy -- have served to accommodate this need and thereby open up the possibility for an almost unlimited range of ownership interests.

FERC's liberal policy is now more crucial than ever as many projects conceived since 1980 have matured to the point of needing financial as well as managerial assistance. Some of these have been constructed and are in commercial operation, some have been licensed and are still seeking the means to undertake construction, and some have not yet been licensed because of ongoing complexities.

Representative Examples of Evolving Project Ownership

An examination of four recent actual case histories will illustrate the opportunities presented, for developers and investors alike.

Case No. 1: The New Martinsville Project. Located at the U.S. Army Corps of Engineers' Hannibal Lock and Dam on the Ohio River, this 34-Mw project was licensed in 1984 and began commercial operation in July 1988. The New Martinsville Project was conceived as an undertaking of the City of New Martinsville, West Virginia and Catalyst Energy Development Corporation. The City of New Martinsville was, and has remained, the sole licensee subject to FERC regulation. Catalyst, in turn, arranged for project financing through a series of transactions involving several other financial entities. Catalyst was the beneficial owner for federal income tax purposes. The financing structure called for all revenues from the Project to be paid over to a Disbursement Trustee, to be allocated first, to pay operating and maintenance expenses; second to pay amounts due under financing documents; third, to fulfill reserve account requirements; fourth, to the extent funds are available, to pay the City a royalty; and fifth, to pay any remaining funds to Catalyst. The project was operated under a service contract arranged by Catalyst but subject to ultimate veto by the City of New Martinsville (a critical feature in view of FERC licensing requirements).

After two years of operation, Catalyst's interest was sold to ERCE Environmental and Energy Services Company, which has since been acquired by Ogden. A new entity also assumed the O&M contract for the project in lieu of the original contractor. The new service company is, of course, still subject to the ultimate control of the City of New Martinsville as the licensee.

In sum, in the case of the New Martinsville Project, the project interests evolved from municipal licensee plus financial party plus service contractor, to municipal licensee plus new financial investor plus new contractor. Thus, a new financial entity and a new operating contractor assumed prior interests and roles previously held by others after the start of commercial operation, all without the need for any additional approval by the FERC subsequent to the issuance of the license.

Case No. 2: The Belleville Project. This Project, also located at a Corps of Engineers' facility on the Ohio River, presents an evolution of ownership and operating interests involving a partial change in licensee status prior to project construction. The City of Jackson, Ohio, was granted a license for this 42-Mw project in 1989. The license application was prepared by Jackson's consulting engineers, pursuant to a contract whereby Jackson could either pay the consultant its fees or share the project ownership. Since Jackson's system load is 14 Mw, Jackson sought to market the Project output which is excess to its internal requirements.

Jackson elected to pay its original consultant its project-related fees in lieu of partial project ownership. With the assistance of American Municipal Power-Ohio, Inc., the state wide municipal electric membership organization, Jackson then enlisted a number of other Ohio municipalities in acquiring ownership interests in the Project proportionate to each city's entitlement to Project power, while sharing project costs. The participating municipalities executed a management services contract with their membership organization, AMP-Ohio, to perform operating and maintenance duties. This has provided both a secure market and an attractive means to finance the Project through the issuance of tax exempt revenue bonds, as well as day-to-day management oversight by the municipals' state-wide organization.

The Belleville Project represents a case of starting with a sole municipal licensee plus private consultant with ownership option, evolving to a sole municipal licensee, further evolving to a group of municipal licensees with a new management services contractor. This evolution was designed to avoid the possibility of exposing the transfer to new competition by maintaining the municipal preference status upon which the original licensing decision was based.

Case No. 3: The Chace Mill Project. Chace Mill, located on the Winooski River in Vermont, was conceived as a joint permit proposal by the City of Burlington Electric Department and Green Mountain Power Corporation, an investor owned utility ("IOU"). At the licensing stage, Green Mountain Power withdrew and Burlington was the sole applicant for a 13-Mw proposal. While Burlington's license application was still pending, Winooski One Partnership entered into an agreement with Burlington whereby both would become owners/licensees, and the project concept was re-configured to a more modest, 6.5 Mw proposal.

The Commission approved the joint agreement, then issued the license to Burlington and Winooski Partnership as co-licensees. While the project was still under construction, Burlington decided to relinquish its current ownership interests altogether, and the license was transferred to Winooski Partnership as sole licensee. Burlington, however, retained a right to re-acquire the project after ten years, subject to any necessary FERC transfer approval which may be required if and when Burlington exercises its option.

Chace Mill thus represents a case of project ownership evolving from a municipal plus investor owned utility as permit holders, to sole municipal license applicant, to municipal plus independent power producer ("IPP") as applicants and as co-licensees, to IPP as sole licensee, with several investors in the entity holding the license. Some of the evolution occurred prior to licensing, while the final stage occurred during project construction.

Case No. 4: The Hudson Falls Project. Another recent example with yet an entirely different cast of entities with different objectives is the 36-Mw Hudson Falls Project on the Hudson River in New York. This project was originally one of several units of development in Niagara Mohawk's Hudson River Project. When the original license term was nearing expiration,

Niagara, at the FERC's suggestion, filed a separate license application for the Hudson Falls Project. This invited competition from Long Lake Energy Corporation, an IPP.

As part of a settlement of extensive litigation between Niagara and Long Lake, the two competitors agreed that another IPP, Adirondack Hydro Development Corporation, would acquire a major interest in the project. Adirondack Hydro, in turn, solicited investments from other equity investors, domestic and international, whose interests were structured so that they need not become licensees. Thus, in the case of the Hudson Falls Project, the interests evolved from: sole IOU license applicant, to IOU plus IPP no. 1 as separate license applicants, to IOU plus IPP no. 2 as prospective licensees with non-licensee equity investors, and will possibly emerge further to IPP no. 2 as sole licensee with IOU and equity investors still involved.

Summary and Conclusions

Multifaceted, creative financial and ownership arrangements have become increasingly necessary by the changes in the FPA and the tax laws referenced above, as well as basic economic considerations. The economic realities of project development often do not become a real factor to many developers with limited resources until after a license is issued.

The varieties of arrangements, of which the four situations summarized above are merely examples, are made possible because of the FERC's liberalized policy concerning project ownership interests. Until 1984, the FERC imposed a fairly rigid view of the requirements of project ownership and licensee status. Then, beginning with the Little Falls decisions when it *first*, reversed its Staff's ruling that even limited partners must be named as licensees, and *second*, ruled that transfer of legal title to project property to an industrial development agency did not require transfer of the license pursuant to Section 8 of the FPA, the Commission has adopted and expanded a more pragmatic, creative policy which encourages an unlimited variety of interests to become involved in projects.

"Control" over the project must remain nominally in the licensee(s), of course. However, the test of what constitutes such control over proprietary interests is far less restrictive than previously. In the City of New Martinsville declaratory order, 7/ the

Commission stated that alternative creative financing arrangements are permissible and that "requests for formal Commission approval thereof are not necessary", so long as the licensees maintain *"sufficient ownership"* and control of a project to enable the Commission to exercise its regulatory responsibilities.

In addition, the FERC has recognized certain contractual provisions, generally referred to as the so-called "Linweave clause" 8/, which will assure regulatory authority over a project while allowing investors who prefer some control over their investment to not become licensees. Such a clause was employed in the Belleville Project contractual arrangements and in some of the documents in the Hudson Falls case to provide additional assurance that certain entities would not be required to become licensees.

Along the same lines, the Commission also has recently endorsed in concept the notion of a *temporary* licensee, by approving a transfer of several licenses to an institutional lender that was forced to foreclose on several licenses to secure its interests on loans. 9/ The transfers were approved notwithstanding the bank's stated position that it did not intend to be the project developer, but rather anticipated subsequently transferring the project to an able developer as soon as one could be found. The FERC also granted a stay pending approval of the transfer in recognition of the "novel" circumstances involving the role of the financial institution. Specifically, in granting the stay, the Commission held that failure to protect the bank's interests "could...have the effect of discouraging financial institutions from committing capital to the development of hydroelectric projects, contrary to the public interest." 10/ The Commission not only approved

8/ The clause takes its name from the Commission's ruling in Linweave, Inc., 23 FERC ¶ 61,391 (1983), in which the Commission imposed a contractual provision which expressly provided the lessee of project property with the right to perform any and all acts required by the FERC. *See also*, New York Irrigation District, 58 FERC ¶ 61,271 (1992).

9/ Power Resources Development Corporation and Jefferson National Bank, 60 FERC ¶ 62,126 (1992); Yankee Hydro Corporation and Jefferson National Bank, 60 FERC ¶ 62,126 (1992); 60 FERC ¶ 62, 126 (1992).

10/ Yankee Hydro Corporation, 55 FERC ¶ 61,357 (1991).

this interim licensee status, but also worked with the bank to keep the licenses intact as long as possible. 11/

The economic necessity for such arrangements, coupled with the FERC's receptiveness to them, presents a new age of opportunity for investors and contractors. In New Martinsville, the initial investor could be replaced by a new investor with no FERC involvement, since FERC had already approved a creative, complex arrangement at the time of licensing, and there were no changes in the Project's proprietary interests when Ogden succeeded Catalyst.

In Belleville, AMP-Ohio could become actively involved in project development, and assume the operational repressibilities on behalf of its members. With the protection of a Linweave clause, AMP-Ohio did not have to become a licensee, thereby preserving the municipal status of the licensees during the transfer.

In Chace Mill, the Commission was receptive to a series of changes from private ownership to municipal body to independent developer spanning all stages of the project from preliminary permit through project construction. Similarly, in the Hudson Falls case, while the objectives and structure of the various interests are completely different from those presented in the Chace Mill situation, the change in project interests evolved continually through various stages of the regulatory and development process.

These cases represent only a few of many which evidence the FERC's favorable recognition of the unlimited extent of project ownership interests. In this era of many well-conceived projects in need of assistance due to unforeseen circumstances, these cases send an important and encouraging signal of accommodation to investors and developers alike.

11/ Unfortunately, no developers were willing to proceed with the projects, so the Bank ultimately surrendered the licenses.

HYDROPOWER STUDIES IN THE GENESEE RIVER BASIN, NY

Bradford S. Price, P.E., Member (1)

Abstract

Numerous studies have been accomplished since 1836 of single and multiple purpose water resource development in the 642,317 ha Genesee River Basin, NY One study, completed in 1944, recommended construction of a dam for flood control on the river near the village of Mt. Morris, NY, about 64.4 km upstream from the city of Rochester, NY. The dam, completed in 1952, included penstocks for future hydropower development. Another study, completed in 1988, presented results of multiple purpose reservoir studies of the existing Mt. Morris dam and at potential reservoir sites near Portageville, Stannard and Dansville, NY.

This paper presents a summary of the historical and recently completed studies. The need for additional studies of multiple purpose water resource development, including hydropower, at the Mt. Morris dam will also be presented. The views expressed in this paper are the authors and do not necessarily reflect those of the U.S. Army Corps of Engineers (Corps).

Physiography

The Genesee River Basin: (1) is located in the west-central portion of New York State; (2) is elliptical in shape with a north south axis of about 161 km; (3) has a maximum width of about 64 km; and, (4) has an area of approximately 642,317 ha. A basin map is shown on Figure 1.

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(1) Civil Engineer, Project Management Policy and Development Branch, CECW-LP, HQ, U.S. Army Corps of Engineers, 20 Massachusetts Ave. N.W., Washington, D.C. 20314-1000

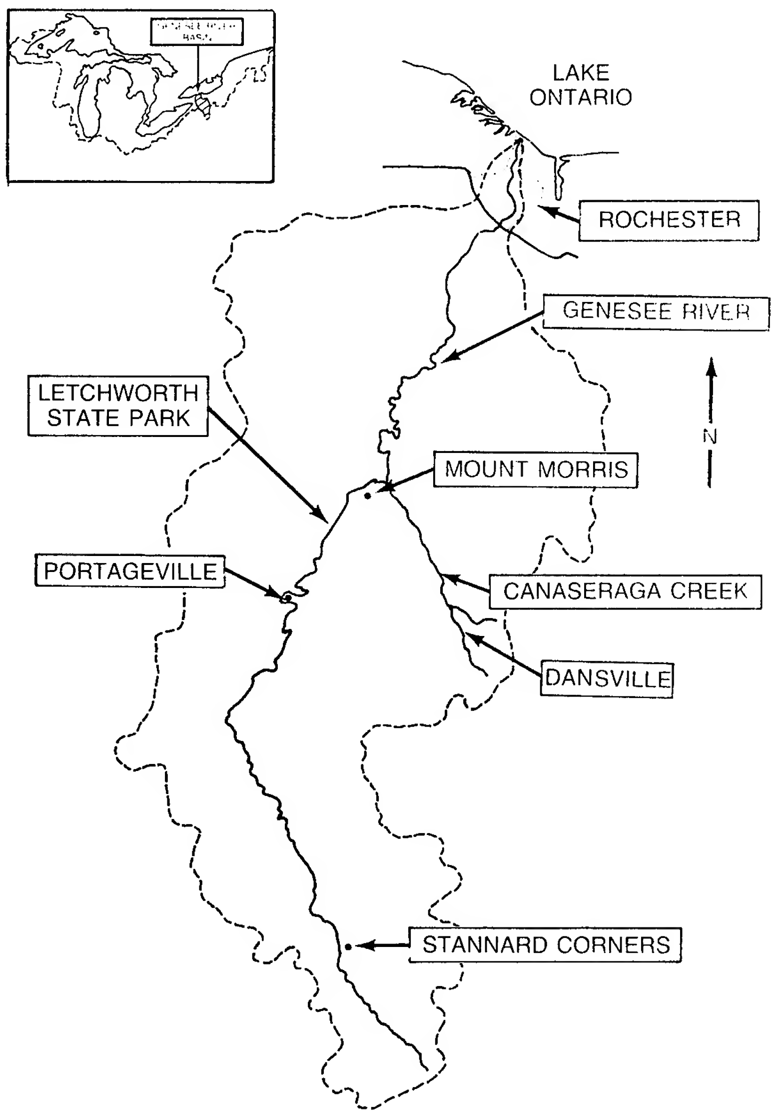

FIGURE 1-GENESEE RIVER BASIN MAP

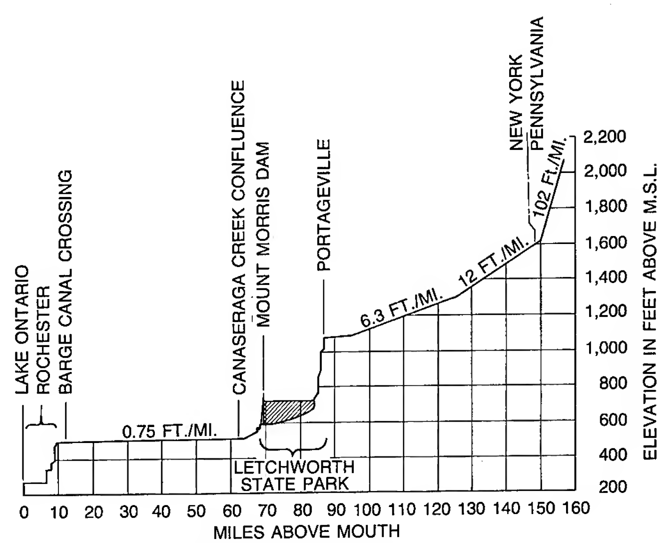

FIGURE 2-GENESEE RIVER BASIN-MAIN STEM PROFILE

The southern portion of the basin, upstream from the village of Mt. Morris is steep and rugged. The northern portion downstream is gently rolling plains. Near Portageville, NY, the river flows over a series of three falls with a drop of approximately 95 m and through a deep gorge in Letchworth State Park. At Rochester, NY, the river flows over another series of three falls with a drop of approximately 71 m and enters Lake Ontario. Profiles of the main stem of the Genesee River are shown in Figures 2 and 3.

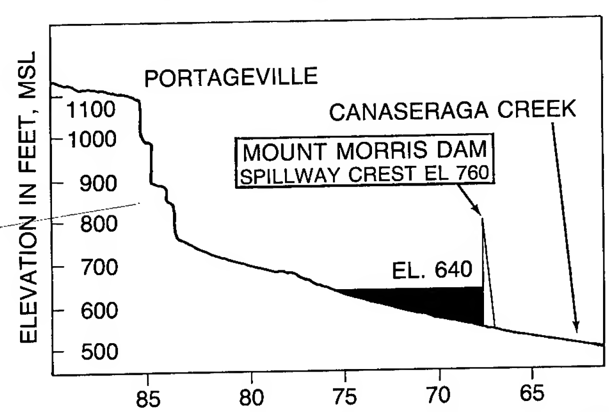

FIGURE 3-GENESEE RIVER-MILES FROM LAKE ONTARIO

Summary Of Historical Projects, Reports and Investigations

Records indicate that water power was first developed on the Genesee River in 1826 at a dam located just upstream of the present Route 36 bridge in the village of Mt. Morris, NY. This facility, currently referred to as Station 160, is operated as a run-of-river hydropower project. Three other run-of-river hydropower projects were developed in the early 1900's on the river at the three falls in Rochester, NY. Data for the four existing projects, which are presently owned and operated by the Rochester Gas and Electric Co. (RG&E), is presented in Table 1.

TABLE 1-RG&E CO. HYDROPOWER PROJECTS ON THE GENESEE RIVER, NY EXISTING (ULTIMATE) DEVELOPMENT

STA. #	RIVER KM	MIN. USABLE FLOW CMS	MAX. USABLE FLOW CMS	NET HEAD M	PF %	NAME PLATE RATING KW	AVG. ANN GEN. MWH
5	6.44	5.7	121.8	39.5	49	38,250	165,000
			(141.6)		(46)	(43,875)	(177,000)
2	8.45	13.7	35.4	26.2	77	6,500	44,000
		(15.5)	(39.6)				(47,000)
26	9.66	5.7	50.9	7.6	60	3,000	12,000
		(7.1)	(53.8)		(62)		(18,000)
160	109.44	3.9	9.6	5.8	97	340	1,000
					(78)	(420)	(2,900)

In 1836, the New York State Legislature authorized construction of a canal along the Genesee River between the Erie Canal near Rochester, NY and the Allegheny River, PA. Construction for 106 locks began in 1837 and lasted 21 years. The canal closed in 1877 due to increased railroad usage.

Between 1889-1893, the State of New York investigated the feasibility of reservoirs on the Genesee River for water supply for the Erie Canal. Sites within the gorge upstream from the village of Mt. Morris, NY were investigated but not developed due to developments at other locations.

The Water Supply Commission of the State of New York, between 1907 and 1910, made a study of the Genesee River for flood control and power.

Two sites were found for multiple purpose reservoirs, one near the village of Mt. Morris, NY and the other near Portageville, NY.

In the 1920's, the Mt. Morris Water Power Company (Company) developed a plan for a dam with hydropower capability across the Genesee River upstream from the village of Mt. Morris, NY. Some of the lands acquired by the Company that were excess to their needs were conveyed to the state of New York for use as parkland in perpetuity on July 12, 1926 in accordance with Chapter 379 of the state's laws. In return, the Company received the right to vary and control flow in the Genesee River subject to the condition that the water level maintained not exceed a 231.6 m elevation for a mile upstream of the dam. The RG&E Co. subsequently purchased the assets of the Company and maintained interest in the Mt. Morris site for hydropower.

A preliminary examination and survey for flood control on the Genesee River was authorized in June 1936. A survey report, published in 1944 in House Document No. 615, 78th Congress, 2nd session, recommended construction of an earthfill dam upstream from the village of Mt. Morris, NY. The dam was authorized for construction by Section 10 of the Flood Control Act, Public Law 534, 78th Congress, approved 22 December 1944. A concrete gravity dam with overflow spillway was constructed by the Corps between March 1948 and May 1952. Penstocks were constructed in the left abutment for future hydropower development. Construction costs were approximately $23 million. Cumulative flood damages prevented through 1991 are estimated at over $380 million.

A comprehensive water resources study of the Genesee River Basin, authorized in 1962 by the Committee on Public Works of the United States Senate, was completed by the Corps in 1969. The study detail was insufficient for project authorization. The Final Level B Study Report, completed in 1970, contained recommendations as a guide to future development. An early-action plan included a multiple purpose reservoir at the Stannard, NY site located on the Genesee River south of Wellsville, NY. The level B Study also examined the multiple purpose Portage Reservoir Project which would have served hydropower and other needs but was deferred because of local opposition. The level B study found streambank erosion along the main stem of the Genesee River to be widespread, but erosion control using individual projects was not economically feasible.

The "Mount Morris Storage Allocation Study" authorized by Section 214 of the 1965 Flood Control Act and completed in September 1971 by the Corps concluded that Mt. Morris Reservoir had excess flood control storage at certain times of the year which could be used to supply conservation purposes without measurably reducing its level of flood protection. Further studies were recommended.

In November 1977, the New York State Department of Environmental Conservation and the Genesee River Basin Regional Water Resources Planning Board published the "Comprehensive Water Resources Plan for the Genesee River Basin". Basic elements of the plan placed emphasis on existing needs and problems and proposals including improvement of water quality, an accelerated flood plain management program, and improved multiple purpose management of lakes, the Barge Canal, and Mt. Morris Reservoir.

The Federal Energy Regulatory Agency (FERC) completed a study of "Headwater Benefits" pursuant to the Federal Power Act in October 1985. The study was made to determine increased energy generation at the RG&E Stations 5,2 and 26 located near Rochester, NY due to the operation of the Mt. Morris dam. Water that would have spilled at the RG&E Stations during floods without the Mt. Morris dam is now stored at Mt. Morris and slowly released after flooding has subsided but at rates that are higher than would have occurred naturally. This allows the RG&E Stations the opportunity to generate additional electricity.

The Headwater Benefits study determined that the operation of the Mt. Morris dam increased the energy output from these Stations by an average of 7,530 mwh annually and that the RG&E should be assessed $1,225,050 for benefits received from 1952 through 1983, plus $38,732 for cost of FERC studies and $54,599 for Headwater Benefits received in 1984, for a total of $1,318,381. Between 1984 and 1991, an additional $257,922 has been collected for a grand total to date of $1,576,303. These funds were deposited in the Federal Treasury.

The average annual energy generation of 7,530 mwh represents approximately 3.4% of the total average annual energy generation for the three RG&E Stations. There are additional increases that could be realized through, at a minimum, changes in the operation of Mt. Morris as will be discussed later in this paper.

Recently Completed Studies

As previously described, the Corps completed a comprehensive investigation of the Genesee River Basin in 1969 followed by more detailed studies in 1970. This resulted in a recommended early action plan that was deferred due to a lack of strong local support.

In Fiscal Year 1985, funds were provided to resume the studies. A Reconnaissance Report published in August 1986, recommended detailed studies of the considered dam and reservoir at Stannard, NY for flood control and other uses; re-regulation of the existing Mt. Morris dam reservoir outflows; and addition of gates on top of the spillway section of the existing Mt. Morris dam for increased flood protection. A number of alternatives

were eliminated due to a lack of economic justification and potential for adverse environmental impacts. Two of these included reservoirs at Dansville, NY and Portageville, NY

The Feasibility phase of the study was initiated in 1986. Consistent with Corps policies at the time, the primary water resource need for which solutions were sought was to reduce flood damages in the basin. Five alternative plans were evaluated during this stage of study and include:

1. The addition of 27-foot high tainter gates onto the top of the spillway section of the existing Mt. Morris Dam for additional flood control and for irrigation, recreation, and hydropower.
2. A dam and reservoir at the Stannard, NY site, for flood control and recreation.
3. The construction of a dam and reservoir at Stannard, NY and addition of 27-foot high spillway gates onto the existing Mt. Morris dam for flood control, hydropower generation, and irrigation of the area to the west of Rochester, NY.
4. The re-regulation of the existing Mt. Morris reservoir to reduce the rate of erosion of downstream channels and to reduce agricultural flooding.
5. No Federal action.

Plans that included the Stannard, NY reservoir were subsequently eliminated mainly because of the lack of economic justification for single-purpose flood control, local opposition and environmental impacts. Multiple purpose plans at the Mt. Morris dam would have met the additional flood control needs and provided for further development of hydroelectric power, recreation and increased irrigation. The economic analysis for these plans indicated benefit-to-cost ratios above 1.0 for multiple purpose use.

However, given the social and perceived adverse environmental impacts of such alternatives, a number of residents basin-wide expressed opposition to these alternatives. Further, the probability of implementing these multiple purpose plans was very low as the Corps and the New York State Department of Environmental Conservation could only implement the flood control component of the plans. The irrigation and recreation components were dropped from further consideration because of the lack of non-Federal sponsors to share in the costs of additional studies, cost of construction and cost of operation and maintenance of any potential projects. Overall, multiple purpose plans were given little further consideration.

Remaining Feasibility study efforts focused on a combination of Plans 1 and 4 above but only for flood damage reduction. The re-regulation plan would result in periodic pools that are temporary in nature as they vary with the inflows to and outflows from the reservoir. The second component of the plan included tainter gates on top of the spillway section of the dam. Plans were formulated that included gates of 3.66 m, 6.71 m and 9.14 m in height. The tainter gates would provide for use of additional storage in the

reservoir and hence, additional flood protection for all flood events including Tropical Storm Agnus that occurred in June 1972. Tables 2 and 3 present summary benefit and cost data for these plans. The Corps concluded that additional study of these plans was not warranted due to the lack of economic justification. The Feasibility study results were included in the Corps report dated June 1988.

TABLE 2-AVERAGE ANNUAL BENEFITS - MT. MORRIS DAM (SPILLWAY GATES PLUS RE-REGULATION)

BENEFIT CATEGORY	3.66 METER GATES	6.71 METER GATES	9.14 METER GATES
RESIDENTIAL	$137,400	$207,700	$327,200
COMMERCIAL	$149,700	$222,900	$307,200
MUN. & UTIL.	$87,300	$127,700	$164,400
AGRICULTURE	$117,000	$158,500	$222,700
EROSION	$2,000	$2,000	$2,000
TOTAL	$493,400	$718,800	$1,023,500

NOTE: All costs and benefits are based on January 1988 prices, interest rate of 8-5/8%, and 100-year project life.

TABLE 3-ECONOMIC EFFICIENCY - MT. MORRIS DAM (SPILLWAY GATES PLUS RE-REGULATION)

CATEGORY	3.66 METER GATES	6.71 METER GATES	9.14 METER GATES
TOTAL AVERAGE ANNUAL COST (1)	$942,800	$1,127,600	$1,266,100
TOTAL AVERAGE ANNUAL BENEFIT	$493,400	$718,800	$1,023,500
NET BENEFITS	$-449,400	$-408,800	$-242,600
B/C RATIO	0.5	0.6	0.8

NOTE: All costs and benefits are based on January 1988 prices, interest rate of 8-5/8%, and 100-year project life.
(1) Does not include the cost for raising the top of the non-overflow sections of the dam by 3.96 m to accommodate the Spillway Design Flood.

Assessment Of Recently Completed Studies

The Corps Feasibility studies were conducted consistent with current Corps policies at the time. The primary water resources need for which solutions were sought under the Congressional authority being utilized was to reduce flood damages in the Genesee River Basin. This limited the Corps ability to fully investigate other water resource needs including hydropower, recreation, irrigation, etc. because of the lack of non-Federal partners to share in the cost of these additional studies as required by the Water Resources Development Act of 1986.

Consequently, Corps studies eliminated most plans that included multiple purpose use and even concluded that modifications to the Mt. Morris Dam were not feasible for flood control purposes only. However, the economic analysis for plans that included multiple purpose use indicated incremental benefit cost ratios above one for each purpose.

Additionally, there was genuine concern raised by the public about the impacts the addition of tainter gates and changes in storage utilization behind the Mt. Morris dam would have on the scenic and environmental attributes of the river gorge between the dam and the three falls upstream. The Corps had few resources, because of the lack of partners to share in study costs, to examine these impacts to any great extent and address the concerns of the public.

Therefore, additional studies are needed to determine, at a minimum, the economic viability of multiple purpose water resource use of the Mt. Morris dam and reservoir. Also, detailed environmental and social studies are needed to ascertain the impacts of any considered modifications. These studies are needed to ensure that a reasonable opportunity is provided for potentially implementable plans to be given fair consideration.

Measures To Be Considered In Future Studies

Measures that should be considered in future studies and the rationale for their consideration include:

1) Modify The Current Outflows From Mt. Morris-This measure should be examined individually and in combination with other measures. Currently, the dam is operated during and following flood events to draw the reservoir pool down as quickly as possible. Alternative operating scenarios should be examined that include reducing the drawdown rates to reduce downstream channel erosion and hence reduce sediments to be dredged from Rochester Harbor and to enhance hydropower output at downstream RG&E Stations (Headwater Benefits). The joint probability of sequential flood events should be considered in the evaluation.

2) Seasonal Flood Control And Conservation Pool-This could be examined with or without tainter gates on the spillway crest. The primary

purpose would be to provide a fairly stable summer recreation pool behind Mt. Morris for use by visitors to Letchworth State Park (Figure 3). A secondary purpose would be to provide a head for a run-of river hydropower project that includes bifurcation's, a couple of the flood control tunnels through the dam and construction of a small power house in the vicinity of the dams stilling basin.

3) Adding Tainter Gates To Mt. Morris Dam Spillway Crest-The Corps feasibility studies focused on this measure but only for flood control, streambank erosion and irrigation. Additional studies should include: (1) consideration of water based recreation at and downstream from the dam; (2) hydropower at the dam; (3) headwater benefits and development of additional hydropower capacity at downstream RG&E Stations; and, (4) downstream instream water quality, recreation and other needs.

Study Authorities

The Corps may be able to conduct studies associated with modifying the operation of the dam under their existing operation and maintenance authority. The state of New York and the RG&E Co. could serve as non-Federal sponsors based upon their interests in stream bank erosion and headwater benefits, respectively.

The Corps has the authority to investigate the other measures described above under Section 216 of the Flood Control Act of 1970, Public Law 91-611, but may need additional Congressional authorization for construction. Also, non-Federal sponsors would have to be identified to share in some of the study costs and any construction costs. The details of cost sharing requirements would have to be identified during the early study stages.

Individual agencies could be identified to cooperate as non-Federal sponsors or a single agency could be established to serve as a primary sponsor and coordinate the efforts of the other non-Federal interests. This agency, referred to as a River Regulating District could be established in accordance with Title 21, Article 15-2103 of the New York State Environmental Conservation Law.

Conclusion

Numerous studies of the potential for water resource development in the Genesee River Basin have been accomplished over the years. Additional studies should be accomplished to determine economic viability in more detail for a number of measures including modified operation of the existing Mt. Morris dam and the addition of tainter gates to the spillway crest of the dam. Additional studies are also needed to determine the detailed social and environmental impacts of implementing any of these measures.

Appendix 1. References.

1. Federal Power Act (41 Stat. 1063, 16 U.S.C. 791-823) 10 June 1920, as amended (FPA).

2. FERC, Staff Report, Investigation Of Headwater Benefits In The Genesee River Basin, 1952 Through 1983, Docket No. HB62-84-1000, October 1985.

3. R.G.&E. Co., Application For FERC Major License Renewal, Project #2582., Station #2, December 1991.

4. R.G.&E. Co., Application For FERC Major License Renewal, Project #2583, Station #5, December 1991.

5. R.G.&E. Co., Application For FERC Major License Renewal, Project #2584, Station # 26, December 1991.

6. R.G.&E. Co., Application For FERC Minor License Renewal (Less Than 5mw), Project #2596, Station #160, December 1991.

7. U.S. Army Corps of Engineers, Buffalo District, Feasibility Report, Genesee River Basin Study, June 1988.

8. U.S. Army Corps of Engineers, Buffalo District, Phase 1 Report On Mount Morris Storage Allocation Study, September 1971.

9. U.S. Army Corps of Engineers, Buffalo District, Reconnaissance Report Genesee River Basin Study, August 1986.

Deciding Competing Resource Use Issues at FERC—From Theory to Practice

James M. Fargo[1]

Abstract

By the close of the December 31, 1991, deadline, the licensees of 158 hydroelectric plants, totalling roughly 2,000 megawatts of installed capacity, had filed applications for relicense with the Commission. Though we on the Commission staff will now need to consider many aspects of each proposal, the most controversial aspect we'll face is how to choose among competing uses of each waterway.

In this paper, I'll present both the theories that guide how we select among applicants' proposals and alternative proposals and the methods we use to make these choices.

After discussing the theory and methods, I'll give examples from two recent environmental assessments to show how we choose the license option that we think gives the greatest benefit to the public. This way, you'll see how we apply these methods, what aspects are important, and what difficulties we face in making these choices.

Introduction: Considering All Goals Equally

In 1986, the Electric Consumers Protection Act amended the Federal Power Act (Act), requiring the Commission to give the environment—including recreation, fish, and wildlife resources—equal consideration with power and other developmental goals, such as irrigation, water supply, and flood control. Since the Commission already had the responsibility under Section 10(a) of the Act to consider all aspects of the public interest, ECPA doesn't add any new waterway uses to what the Commission must look at. But it does raise the question of how the Commission can give all aspects of a proceeding "equal consideration."

1 Chief, West Engineering Section, Division of Project Review, Federal Energy Regulatory Commission.

In license orders issued after ECPA,[2] the Commission says to consider developmental and nondevelopmental goals equally they need a record that fully evaluates all developmental and nondevelopmental aspects—including the arguments of agencies and interveners on competing uses of the water resource.

Deciding Among Competing Uses of a Resource: the Theory

While we now need to consider all aspects fully in each proceeding, no specific policy or theory guides us in how to decide among competing resource uses. So, to make these choices, we often use an approach widely used by state and federal resource managers: we compare the economic value of each alternative use (Edwards, 1991).

Edwards defines economic value as a measure of the total pleasure that people derive from goods and services (including natural resources). Economic value is usually measured by the highest amount consumers are willing to pay for a good or service.

When we lack information on what people are willing to pay for environmental goods, looking at how various water allocations would affect economic value can often help us decide the best use.

As an example, let's look at how we might use economic value in deciding the best proposal to raise instream flow in a bypass reach. Besides putting a value on the offstream power use of flow, we'd attempt to answer the question of how much flow is worth when left in the particular stretch of river.

If we can't express the instream value in dollars, by quantifying how each proposal would affect the stream and deciding how significant the effects would be to the environmental resources and to the public, we could express the value in relative terms. Once we determine these relative economic values, we can use them to compare a proposal to other instream flow proposals or to the relative economic value of other streams.

To choose the best use of a resource in some proceedings, we look at economic value and we also consider other aspects—such as how alternative instream flows affect an Indian Tribe's fishing rights or the religious value a Tribe holds for a fishery.

So when we decide among competing proposals, we must deal with both (1) resource effects we can quantify, often expressed in dollars, and (2) aspects we must describe qualitatively, sometimes based on expert opinion.

In proceedings with qualitative aspects, we usually look at the economic value of the proposals first. Then we see whether the best qualitative proposal differs from the best economic proposal.

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2 *Brazos River Authority*, 48 FERC ¶ 62,190 (1989).

So for the instream flow example, we'd first find the best proposal based on the economic value of power and the instream uses of flow. Then, we'd decide whether any qualitative aspects we must consider—such as how each proposal would affect Indian fishing rights—changes the instream flow we'd recommend on the basis of economics.

Using Economic Value to Choose Among Competing Interests

As I've said, to consider competing proposals to allocate a resource—such as proposals to set instream flows—we often use economic principles to guide us: by looking at the economic values of a range of proposals, we try to find the proposal that would give the greatest total value from the resource.

To show how we can use economic value to decide among competing resource uses, let's look at an example from Edwards where he shows the correct way to use economic value to split a fishery between seafood and sportfishing sectors. When different uses conflict, Edward's emphasizes that resource managers should allocate a resource by finding where the net economic value[3] of the next increment of the resource is the same for each competing use.

In his example (figure 1), Edward's shows how allocations of the fishery between the two sectors differ. Using dollars, Edwards calculates a net economic value for various seafood and sportfishing sector shares of total allowable catch. To get the greatest economic value, Edwards allocates the fishery so the value of the next increment of the fishery is the same for either sector.

Looking at figure 1, you cán see that if a manager allocates the total catch of 12 million pounds to the sportfishing sector, the net economic value of the catch would be about $56 million; allocating the total catch to the seafood sector, the net economic value would be about $48 million. So, if a manager had to allocate the whole catch to one sector or the other based on net economic value, he'd pick the sportfishing sector.

Though the sportfishing sector would get more net economic value from the total catch than the seafood sector, figure 1 shows that this allocation doesn't yield the greatest net economic value: splitting the catch between the two competing sectors increases the net economic value to $90 million.

How does dividing the resource between these two competing uses increase the total net economic value?

Going back to figure 1 and looking at the sportfishing curve, we can see that the left side of the curve is very flat. Because of this, the net economic value in the sportfishing sector decreases very little for increases in the seafood catch up to about 6 million pounds. Above 6 million pounds, the sportfishing net economic value decreases at a rapid rate with increases in seafood catch.

3 Net economic value is the difference between total economic value and total resource costs.

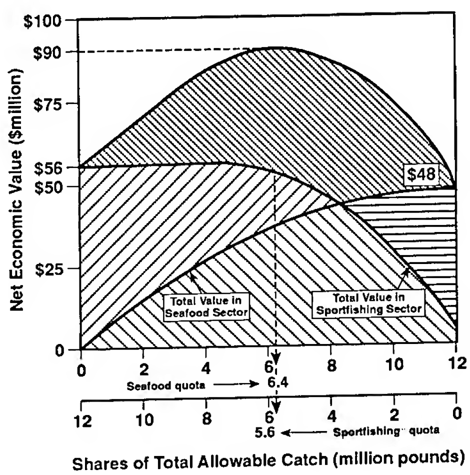

Figure 1: Net economic value from allocation of the 12-million-pound total allowable catch quota between the seafood and sportfishing sectors.

Looking now at the seafood sector curve in figure 1, we can see a steady gain in net economic value as the seafood catch increases.

If we begin with an allocation that gives all the catch to the sportfishing sector, we can see how the net economic value changes as we allocate some catch to the seafood sector. Increasing the seafood catch from 0 to 6.4 million pounds, we lower the sportfishing value slightly but increase the seafood value from $0 to about $38 million. So, without significantly lowering the value of the sportfishing sector, we greatly increase the total net economic value of the resource—increasing net economic value from $56 million, when we give all the catch to sportfishing, to $90 million when we split the catch between the two sectors.

If we look at the slopes of the seafood and sportfishing curves at a seafood allocation of 6.4 million pounds, we can see the slopes are about equal. So, at this allocation, an increment of catch has the same net economic value to either sector.

As Edwards says, this general principle—getting the greatest net economic value by making the net economic value of an increment of a resource the same for each competing use—holds whenever we compare the economic values of competing uses, whether the uses are fishing and waste disposal, oil production, habitat destruction, aquaculture, or species preservation.

If we apply these same economic principles to allocate water between an instream fishery and a hydroelectric plant, to get the greatest economic value,

we'd try to allocate the flow so that the next increment of water has the same value to both the fishery or the powerplant.

So the greatest economic value probably wouldn't come from giving all the water to power or all the water to a fishery: it may result from giving some water to a fishery and some to power, up to the point where the next increment of water, if added to the fishery, wouldn't do its share in benefiting the fishery.

Edwards says it's conceivable that a manager would need to eliminate a resource use to get the greatest net economic value from a resource. For two competing resource uses, a manager would need to eliminate a use when the net economic value of the next increment of a resource is greater for one use than the other, throughout the range of possible allocations.

For an instream flow allocation, this need would occur when a small amount of water benefits one resource—fish or power—more than the other, throughout the range of interest.

The Methods: Using Principles of Economic Value

We've now seen that, when we don't have specific policies or theories to rely on in choosing among competing resource uses, we can use two general economic principles:

1. By looking at a range of proposals to allocate a resource, we try to find one that derives the greatest net economic value.
2. To get the greatest net economic value, we allocate a resource so that the next increment of the resource has the same value for each competing use.

When we look at competing resource uses in our license proceedings, the available information we have on each resource influences what method we use to make these choices.

If we can assign dollar values to each affected resource, we can use economic principles alone to compare proposals. When we can't assign a dollar value to a developmental or nondevelopmental aspect, we must use other ways to describe how competing proposals affect it.

In Fargo (1991), I talk about the methods we now use to decide among competing resource uses—(1) assigning a dollar value to all resources, (2) assigning both dollar and nondollar values, and (3) combining the methods. I also talk about the baseline we use to compare alternatives and how many alternatives we need to consider to provide a reasonable range to choose from.

Though each method differs, all three methods are based on these two principles of economic value. By using these tradeoff methods, we can handle the range of competing proposals we must deal with, including (1) resource effects we can quantify and (2) aspects we must describe qualitatively.

The Practice: Our Choices in Two Recent Environmental Assessments

To show how we use these three tradeoff methods in practice, let's look at two projects where we decide among competing resource uses: Beaver City and Crane Valley. The choices we make in both examples are difficult: we choose between instream and power use of flow in Beaver City and we choose among power, fishery, and recreational boating use of flow in Crane Valley.

In these examples, we follow both economic principles I've discussed. We first look at a range of proposals to raise instream flow. Then, we choose an alternative that best allocates flow, consistent with the second economic principle: the value of the next increment of the resource—stream flow—is about the same for each resource use.

Beaver City Project

In our Beaver City Project Environmental Assessment, we look at the effects of continued operation of the project and ways we could enhance environmental uses of the resource. Of the measures we consider, the main issue of competing uses is whether to raise the instream flow in the 2.2-mile-long bypass reach. We expect raising instream flow would improve the associated recreational fishing opportunities.

Currently, when the diversion dam isn't spilling during the spring and summer runoff, the bypass gets only 0 to 3 cubic feet per second (cfs). So increasing the streamflow would increase habitat for the bypass fishery—including all life stages of rainbow and brown trout found in the stream reach.

From our review of recent fish studies, we believe the current bypass flows contribute to lower production levels of fish. Fish population studies and habitat evaluations show that the bypass reach supports 36 pounds per acre of wild trout, whereas other sections of Beaver River, which are not affected by diversions and have similar habitat, support 89 pounds per acre.

Defining the range of alternatives: Besides Beaver City's proposal to release 6 cfs year-round in the bypass, the Forest Service, Utah State Department of Natural Resources (DWR), and the U.S. Fish and Wildlife Service (FWS) proposed raising the bypass flow. With each agency instream flow proposal, the bypass flows varied, depending on the time of year.

Using information from Beaver City's habitat simulation model, we saw how habitat for rainbow and brown trout varies with instream flow. Looking at the habitat each proposal would provide, we decided that Beaver City's and the agencies' proposals gave us a reasonable range of enhancement options for the bypass, so we didn't add any alternatives of our own.

Raising instream flow would reduce the project's power generation and revenues but would benefit the fishery, the stream channel, riparian vegetation and other instream resources. Here I show how raising instream flow affects the

two principle competing resource uses—power and the fishery—and how we decided which proposal would give the public the greatest value.

Power: Raising instream flow would reduce the existing net annual benefits from the project as follows: $58,000 with Beaver City's proposal, $75,000 with FS's draft proposal, $81,000 with DWR's proposal, and $94,000 with FWS's proposal.

Fishery: Currently, reduced flows during the winter, late summer, and fall limit the bypass fishery. Using results from Beaver City's habitat simulation model, we looked at the habitat that would be available during these periods with each instream flow proposal.

Finding the greatest value: Because we didn't think we could express the fishery value of each instream flow proposal in dollars, we used the dollar and nondollar method to find the best tradeoff. Using this method, we compared the loss of annual power benefits with changes in rainbow and brown trout habitat by lifestage. In figure 2, we show these losses in power benefits and changes in weighted usable area (WUA) for each species of rainbow trout under each instream flow proposal.

Figure 2 shows that Beaver City's proposal gives the greatest habitat gain per dollar loss in power benefits. Yet, because Beaver City's proposal would provide minimal overwintering and spawning habitat, we didn't think their proposal would significantly improve the fishery.

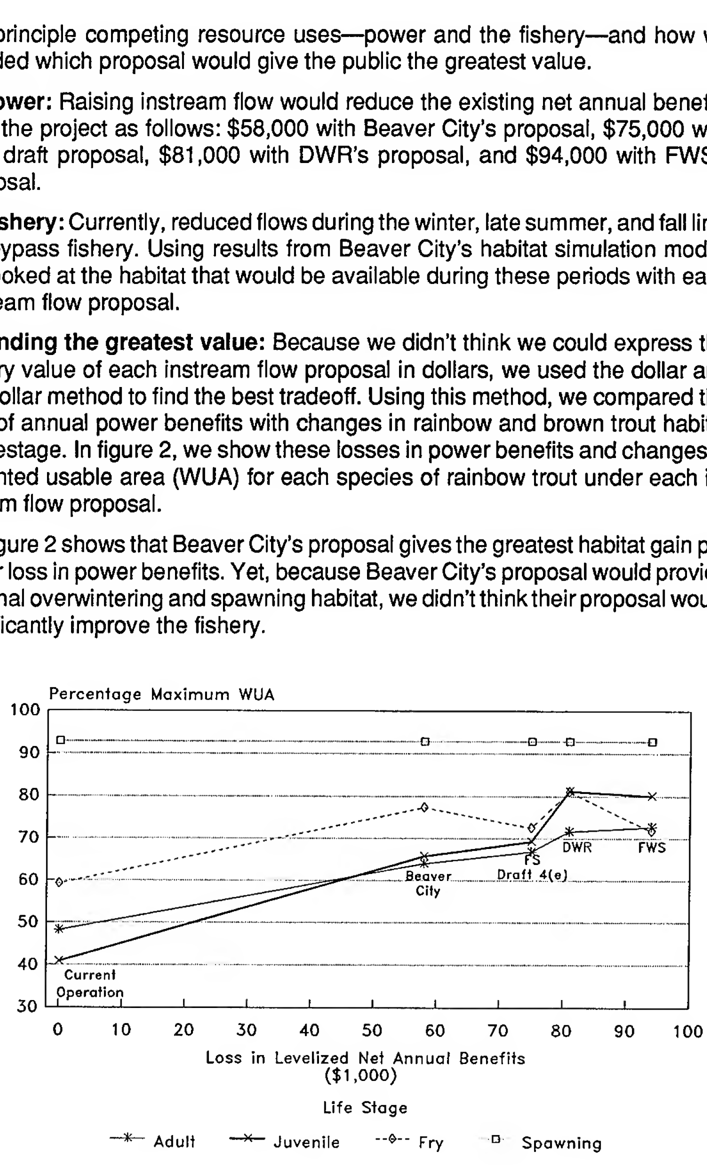

Figure 2: Effects of instream flow alternatives on rainbow trout habitat and power benefits. (Each column of markers represents a flow alternative.)

Comparing Beaver City's and FS's proposals shows that the FS alternative would increase fish habitat over Beaver City's proposal, but at a significant cost ($17,000 annually).

Besides giving "good to optimal" habitat for spawning and fry lifestages, DWR's proposal gives significant gains over both FS's and Beaver City's proposals for overwintering trout. (Figure 2 shows the gains for rainbow trout: adults, 16 percent; juveniles, 33 percent.) We conclude that the DWR proposal, though costing $23,000 more than Beaver City's, would produce enough habitat gain to support a good, self-sustaining fishery.

Looking at the FWS proposal in figure 2, we can see that the gain in WUA between FWS's and DWR's proposals is small, compared to the loss in net annual benefits ($13,000 annually).

For these reasons, we recommended DWR's alternative. With less flow than DWR recommends during the winter months (Beaver City's and FS's proposals), we didn't think the fishery would benefit; with more flow (FWS proposal), we didn't think the increase in fishery benefits would justify the loss in power value.

Crane Valley

In the Environmental Assessment for the relicense of the Crane Valley Project, we look at how raising instream flow for the 5.1-mile-long South Fork Willow Creek bypass reach would affect the fishery, the project's power value, and recreational boating on the project's reservoir, Bass Lake. To develop our recommended instream flows, we combined the tradeoff method that assigns dollar values to all resources and the method that assigns both dollar and non-dollar values.

Defining the range of alternatives: No instream flow requirements now exist for the project. So increasing the streamflow would increase the amount of habitat for all life stages of rainbow and brown trout found in the stream reach.

Besides the instream flow proposal of the applicant, Pacific Gas and Electric (PG&E), we also had a joint agency recommendation from the Forest Service (FS) and the California Department of Fish and Game.

Using information PG&E developed from habitat simulation models, we looked at how available habitat for rainbow and brown trout varies with streamflow. We concluded the instream flows represented by the existing conditions, the applicant's proposal, and the agencies' proposal gave a reasonable range of fishery enhancement options. So we didn't consider it necessary to develop any of our own instream flow alternatives.

Raising instream flow in the South Fork Willow Creek affects three resources—the fishery, power, and recreation—so we needed to find how raising the flow affects the value of each resource before we could recommend an instream flow.

Power: Like most projects, we could calculate the change in the project's power value for each instream flow proposal in dollars. I show these effects in table 1.

Fishery: By raising instream flow under either proposal, the fishery would get large increases in adult trout habitat—from 276 to 344 percent (table 1). Since the value of the power loss is also large, we used another approach to assess the value of raising instream flow: taking the loss in power value, we calculated what the fishing use would have to be to produce the same economic value as the power loss.

Using information on the willingness of sportfishers to pay for trout fishing (Brown and Hay, 1980), we found an annual increase in fishing use of 1,294 fishing-days per mile of stream would provide an economic value equal to the power loss under PG&E's proposal. To offset the power loss from the agencies proposal, an annual increase in fishing use of 1,536 fishing-days per mile of stream would be needed.

Boating: With a boatable surface area of over 530 surface acres, Bass Lake gets high recreational boating use. To show the effects of raising instream flow, we used the water surface elevation of Bass Lake. (The water surface of the lake affects the use and value of the lake for boating and other water-related recreation.) Lowering the lake by raising instream flow would affect boating two ways:

- Lower lake levels would mean that fewer boaters could use their boat docks—decreasing the number of visitor days.
- Lower lake levels would make the boating experience less enjoyable because of overcrowding and unsafe conditions.

With the way PG&E must operate Bass Lake, raising instream flow to enhance the South Fork fishery will lower summer water levels in any year with below normal runoff. The reason: in these "dry" water years, which occur about 30 to 35 percent of the time, PG&E uses little or no flow from Bass Lake to generate power from January through August. Because of this, no opportunity exists in these years to reduce generation in the spring and summer to maintain lake levels.

Bass Lake now gets about 660,000 visitor-days of use, with boating accounting for about 21 percent of this use (138,600 visitor-days)—so lake levels directly affect these users. During dry water years, we estimate that the level of Bass Lake will be about 2 feet lower under PG&E's instream flow proposal and 4 feet lower under the agencies' proposal.

Adding the reduction in existing recreational value of the lake during dry years from both (1) decreased visitor-days and (2) the reduced value of a visitor-day, PG&E's proposal would reduce existing recreation benefits by $2,079,000 and

	Current	Flow Alternative PG&E	Agencies
Normal year summer flow, cfs	0	10	16
Total generation, gigawatthours/year	123.3	120.3	119.8
Percent reduction in generation	0	2.4%	2.8%
Lost energy value, $/year	0	$320,000	$380,000
Total adult Rainbow trout habitat (WUA)	1,775	6,676	7,887
Percent increase in adult trout habitat	baseline	276%	344%
Total Rainbow trout spawning habitat	0	1,262	2,504
Loss in Recreational Boating Value	baseline	$695,000	$925,000

Table 1: Effect of instream flow proposals for South Fork Willow Creek on generation, fish habitat, and recreational boating.

the agency proposal by $2,772,000—a $693,000 greater loss in benefits than PG&E's proposal. Table 1 shows this loss on an annual basis.

Finding the greatest value: After finding the value of each resource for each instream flow proposal, we could then compare the gain to the fishery under each proposal to the loss in power and boating value. We did this in two steps. First, we looked at the power and fishery values to see if we could justify the loss in power value by the fishery gain. Since we could, we then added the loss in boating value to the power loss to see if the gains to the fishery justified these added losses.

Comparing the power loss to the gain in fishery value, we find the habitat gains from either PG&E's or the agencies' proposal would be significant. But the agencies' proposal, with higher summer flows and more spawning habitat, would best meet the agencies' goal of creating a self-sustaining fishery on the South Fork.

Based on our estimate of the economic value of the fishery, we find that either PG&E's proposal or the agencies proposal would have an economic value comparable to the loss in power value: considering the high recreational use of the project and the high demand for recreational fishing, we think increases in fishing use of about 1,294 to 1,536 fishing-days per stream mile are reasonable to expect.

So, looking at just the choice between the fishery and power, we'd have agreed with the agency recommended flows—because (1) the flows could produce a nonpower economic value that could offset most, if not all, of the loss in power value, and (2) the agencies' flows would more than meet their goal of maintaining a self-sustaining trout fishery.

Considering power, recreational boating, and the fishery, we found that the loss in boating value during below normal runoff years would be significant under

both PG&E's and the agencies' proposals. To avoid this reduction in recreational benefits, we recommended (1) a lower instream flow than PG&E's or the agencies' proposals under below normal runoff conditions that would keep the reservoir close to historic levels and (2) the agencies' instream flow under normal and above normal runoff conditions.

With the agencies' instream flows in normal and above years and these lower "dry year" instream flows, the recreational value of the project would stay at the existing level, the power value would decrease some, and the fishery value would increase by an amount that justifies the power loss.

Conclusions

I've now shown you how we choose among competing resource uses at the Commission. Though the issues and complexity of each proceeding differ, the three tradeoff methods we use—based on principles of economic value—often help us find the best way to allocate a resource among competing uses.

In the Beaver City and Crane Valley examples, I explained how our tradeoff methods help us determine the value of using each competing resource for each proposal and guide us in choosing the best way to allocate the resource.

These methods give us a framework for making consistent decisions, but because we must weigh and balance often dissimilar information on power and nonpower resources, the final choice is always difficult. As we get better at predicting how proposed environmental measures increase resource value—such as how raising instream flow would increase fishing opportunities—we can better decide among competing proposals.

Acknowledgements

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The Stone Creek Hydro Project - A Case Study
Building a Small Hydro Project on Public Property

Kenneth C. Fonnesbeck, P.E.[1]

Abstract

The Stone Creek Hydro Project is a 12 MW, run-of-river, hydro project located on U.S. Forest Service property within the Mt. Hood National Forest, near Portland, Oregon. The project came under the scrutiny and jurisdiction of many state and federal agencies and local recreation enthusiasts, by virtue of it's location on scenic, easily accessible, public forest property. This situation presented many challenges to private hydropower development.

This paper will present a brief overview of the project and will then concentrate on a nontechnical discussion of the problems and pitfalls associated with pursuing and constructing a hydro project on public property. Solutions to the problems encountered are offered as examples to aid others considering undertaking a public domain project. In particular, the proponent of such a project should plan on encountering unexpected and lengthy delays and scheduling changes, unforeseen costs and organized opposition. Unusual liability and operational concerns are also common. Projects of this type require careful planning and foresight throughout the permitting, licensing, design and construction phases. Realistic project budgets and time schedules are also important. Resolution of these and other problems to be discussed required careful and firm negotiation, patience and perseverance along with unavoidable, periodic doses of money. The Stone Creek Project serves as a successful example, however that this type of project can be built and should be

[1]Civil Engineering Manager, Hydro West Group, Inc., 1422 130th Ave. NE, Bellevue, WA, 98005

undertaken even with consideration for public ownership of the project lands.

Introduction

Construction of a small hydro project on public property presents a hydro developer or utility with challenges and complications uncommon to projects built on private property. Take a minute and put yourself in the place of the public agency or landowner whose property will be the setting for your potential project. You have likely been given a mandate by either the Federal Government or other governing body as to what can and can't be done in overseeing your landholding. A new hydro project most likely was not envisioned when land use or zoning plans were developed. You've found out that it is possible for any group with the necessary commitment and financial capability to tie up your land and construct a hydro project and with limited exception, you can't stop it. In short, you didn't want this "thing" in the first place and now you're damn well going to make it difficult for the group that has forced this "mess" upon you.

This was the situation the Hydro West Group of Bellevue, Washington found itself in pursuing construction of the Stone Creek Hydroelectric Project, which is the subject of this paper; namely construction of a small hydro project on public lands. A nontechnical discussion of the potential problems this situation presents along with brief mention of solutions developed to overcome these difficulties is provided. It is hoped that others considering pursuit of such a project will benefit from our experiences.

Project Description

The Stone Creek Project is a run-of-river, small hydro facility recently constructed on property administered by the U.S. Forest Service; and more specifically, the Mt. Hood National Forest located near Portland, Oregon. This project was developed by Pacific Oregon Corporation for Puget Sound Power & Light Company. Construction began in August of 1991, and the project is expected to begin commercial operation in June of 1993. This project included a 24,700 foot (7528 m) long, welded steel penstock of 72" (1.83m) diameter, a diversion and intake structure with fish ladder, a powerhouse with a single 5-jet vertical Pelton turbine, with a gross head of 700 ft (213 m), and a design flow of 250 cfs (7 cms). The transmission circuit includes both buried and above-ground segments with an overall length of 11 miles (17.7 km).

The diversion structure was sited immediately below the confluence of Stone Creek with the Oak Grove Fork of the Clackamas River. This setting is unique in that it also takes advantage of the existing Timothy Lake Reservoir, a facility operated by Portland General Electric for their downstream hydro operations as well as recreation and flood control.

Agencies which had significant involvement in the permitting and licensing of this project through either the FERC or the State of Oregon's hydro project licensing programs included the following:

1. Federal Energy Regulatory Commission (FERC)
2. U.S. Forest Service (USFS)
3. U.S. Fish and Wildlife Service (USFWS)
4. U.S. Army Corps of Engineers (ACOE)
5. Oregon Dept. of Water Resources (OWRD)
6. Oregon Dept. of Fish and Wildlife (ODFW)
7. Oregon State Parks and Recreation Dept. (OSP)
8. Oregon State Dept. of Environmental Quality (DEQ)

Potential Problems

Permitting and Licensing

The construction of this particular project was complicated by the fact that the State of Oregon has created their own hydro licensing program; an unfortunate development which more states are pressing for. In Oregon, it is intended that the State's license program mimic the FERC license process and provide more direct involvement and review participation by State agencies. Unfortunately, because of timing factors, the Oregon licensing process did not occur simultaneously with the Federal process for our project. What did happen was a costly and involved review proceeding, which with few exceptions, merely duplicated the Federal licensing process and which reopened many license article requirements. Understandably, this was a situation we did not appreciate. The State of Oregon is well known in the Northwest for their zealous protection of the environment, and in almost every state agency we encountered, we experienced an unspoken opposition to this project. At many times in this state permitting and approval process, we reached a point where we were prepared to bring an ominous state ruling or position to the FERC for their intervention. It seemed in these cases, however that the state would concede to FERC if they knew we had reached our final threshold after we reminded them of FERC's dominant status as reinforced by the courts.

Design Requirements

For obvious reasons you are not at liberty to construct project facilities in any manner you desire when you are ultimately granted the opportunity to construct a hydro project on public lands. The location and appearance or aesthetics of your facilities must be consistent with the landowners policies. In our case involving U.S. Forest Service property, that meant we had to be in compliance with their "Visual Resources Plan". I will note that this plan evolved throughout the entire period we worked to obtain the necessary Forest Service approvals. This is typical of vague license article wording stretched to the limit with local interpretation. We were frustrated as well by the arbitrary judgements of the Forest Service's landscape architects who were given the authority to review our designs and require any modifications they desired with no consideration for difficulty, cost or functional use. Their opinions translated into requirements to provide rough-sawn, unstained cedar siding over our entire masonry powerhouse, surface treatments to conceal any exposed concrete with "natural appearing materials" (we used a cobble masonry approach), and screening of facilities with both existing and planted vegetation. These aesthetic requirements added approximately $150,000 in cost to our project which we did not anticipate and which were not specifically required by any FERC license article.

Construction Restrictions - Schedule

The most significant difficulty we faced in terms of scheduling on this project was due to uncertainties created by regulatory agencies. For example, as we finally neared the point when we had secured all required permits and the FERC's permission to begin construction, we were held up by the U.S. Fish and Wildlife Service's and Forest Service's inability to deal with the Endangered Species Act and specifically, spotted owl concerns. Since this project involved the clearing of several acres of old growth timber to construct the diversion structure and powerhouse, we became ensnared in this unresolved issue. Ultimately, we were successful in working around the agencies indecision, but it caused a delay of many weeks. A related sideline to this problem was the fact that the U.S. Forest Service was prevented from selling the cut timber, and they therefore required us to deck and store it in the project vicinity. It will ultimately be used for the Forest Service's own construction activities, trail maintenance and other mitigative uses such as stream habitat structures, rather than be sold as high grade timber for commercial uses.

Liability and Operational Concerns

The siting of a project on public lands brings with it unique complications relating to public access. The normal utility philosophy of keeping the public as far removed from this type of facility as possible had to be compromised for our project. As such, the facilities had to be designed recognizing the fact that people will be around the intake and powerhouse sites. This problem was compounded for the Stone Creek project by it's location in a heavily used recreation area.

Our project facilities were given a careful review for compliance with O.S.H.A. regulations, and the owners and the insurer of the project conducted their own hazard and risk assessment. From this review we picked up things which had been overlooked. For example, motion activated lights and alarm sirens were selected and installed to scare off unwanted visitors from hazardous or sensitive areas. Our intake was "people-proofed" by installing grating or solid covers over all open areas along with additional handrails and walkways to keep people and undesirable materials away from areas where they could do harm to themselves or project facilities. Finally, we worked with the FERC regional office to convince the Forest Service that it was not a good idea to attract people to a hydro intake and were able to persuade them to abandon their plans for picnic and day use areas. The prospect of unwanted visitors was given an unexpected boost by the Forest Service when upon completion of the intake facility, they were so pleased with the appearance and final outcome that they wished to make use of it as an attractive feature for a day camping and picnic area.

Recommendations and Solutions to Problems Presented

To counter the difficulties we experienced in permitting and licensing, the following suggestions are offered:

1) Aggressively and proactively negotiate with all agencies in order to stand a chance of being successful.

2) When your FERC license is ultimately issued, review all license conditions carefully and work to define any ambiguities.

3) Understand that sitting back and idly hoping for a positive outcome, or response from an agency will undoubtedly lead to disappointment and frustration.

4) Concentrate on developing confidence and trust with agency personnel

and this will pay dividends for you as well as for others which will follow in your footsteps.

5) Be cautious of incremental concessions where the eventual sum of "nickels and dimes" ultimately adds up to a substantial amount which would have been objected to more strongly if the whole package of requirements had been presented initially.

6) With respect to scheduling, first plan for agency created delays and do not box yourself into an unattainable goal or completion deadline. You should expect to have last-minute, unexpected obstacles put in your path which must be dealt with, but if your construction schedules and contracts do not allow realistic slack periods, you will add significantly to your cost and frustration by forcing these requirements upon your own people and contractors.

7) To deal with public access and liability problems, design project facilities to keep them as vandal-proof, and safe as possible. Make sure your designers are aware of the great lengths a few people will go to to be destructive or a nuisance.

Conclusion

Constructing a hydro project on public lands requires extra dedication and effort in order to be successful. At times it will seem that every agency you can think of is opposed to you and your goal. Don't forget that hydropower can be a positive use of public property and is truly a renewable resource. Plan for indecision, delays and unexpected costs going into such a project, and work aggressively to overcome opposition and you will minimize your frustration. Projects of this type present unusual and difficult challenges yet can be undertaken and completed successfully as demonstrated by the Stone Creek Hydro Project.

RUEDI HYDROPOWER: AN ECONOMIC AND ENVIRONMENTAL SUCCESS

Dr. F. Robert McGregor, P.E., M.A.S.C.E.[1]
John Musick, Esq.[2]
Mark Fuller[3]
Wayne Chapman[4]
Dr. John R. Sheaffer[5]

Abstract

The Federal Energy Regulatory Commission issued a license in 1983 to the City of Aspen and Pitkin County, Colorado, to construct and operate a 5.0-megawatt hydropower plant (Ruedi plant) on the main outlet works at Ruedi Dam. Ruedi Dam, a 285-foot high structure which stores 102,369 acre-feet of water, was constructed on the Fryingpan River in the Colorado River Basin in western Colorado by the U.S. Bureau of Reclamation as a part of the Fryingpan-Arkansas transmountain diversion project. The Fryingpan River below Ruedi Dam is a gold metal trout stream which experiences heavy recreational use. Special attention was paid during the feasibility studies and the design of the Ruedi plant to the protection of wetlands and in-stream environmental values.

After a review of competitive proposals, a final design, construction and operations contract was awarded to General Electric Corporation. The contract included incentive clauses and bonus awards to General Electric for power production in excess of 18,000,000 kilowatt hours (kwh) per year.

[1]Senior Engineer, Wright Water Engineers, Inc., Denver, Colorado, USA.

[2]Attorney, Boulder, Colorado, USA.

[3]Director, Open Space and Trails, Pitkin County, Colorado, USA.

[4]Director, Neighborhood Housing & Conservation, Cincinnati, Ohio, USA.

[5]President, Sheaffer and Roland, Inc., Wheaton, Illinois, USA.

The project was completed for a capital cost of approximately $4,100,000 and was placed into operation in September 1985. It has produced an average of 18,600,000 kilowatt hours of power per year without compromising the environmental protection values planned for the project.

The Ruedi plant has provided significant economic benefits to Aspen and Pitkin County. The construction of the Ruedi plant enabled the City of Aspen to develop a more diversified mix of supply of electrical power for its service area. The diversified sources of supply have proven to be reliable and is $300,000 to $600,000 per year less expensive than the previous 30-year "All Requirements" supply contract with PSCo. After the debt for the project has been retired, the net benefit will grow to $600,000 to $1,000,000 per year.

The reduced power costs have permitted the City of Aspen and Pitkin County to apply the revenues from retail power sales toward upgrading electric distribution lines, burying overhead power lines and developing other hydropower and water resource projects.

Thus, the Ruedi plant has placed the energy production potential of the BuRec Ruedi Dam into beneficial use in an environmentally sound manner. Furthermore, it has provided Aspen with the opportunity for some energy independence and to diversify the sources for its power supply. This has produced substantial economic benefits in the near term and the long run. The project serves as a model for intergovernmental and public/private industry cooperation.

Project Description

The City of Aspen, Colorado (Aspen) and the Board of County Commissioners of Pitkin County, Colorado (Pitkin County) applied to the Federal Energy Regulatory Commission (FERC) for a preliminary permit to develop hydropower at Ruedi Dam on October 24, 1980. After the completion of feasibility studies, Aspen and Pitkin County filed a license application for a 5.0-megawatt plant (Ruedi plant) with the FERC. The FERC awarded the license for a 5.0-megawatt facility to Aspen and Pitkin County in 1984.

Ruedi Dam is a U.S. Bureau of Reclamation (BuRec) facility which is part of the Fryingpan-Arkansas project (Fry-Ark). The Fry-Ark project is a major water resources development project for the diversion of water from the Colorado River Basin into the Arkansas River Basin.

Ruedi Dam was completed in 1968 and is located just upstream from the confluence of Rocky Fork Creek and the Fryingpan River. It is about 60 miles north of the City of Aspen in Pitkin County and is within the White River

National Forest. The dam is a 285-foot high structure with a storage capacity of 102,369 acre-feet.

The dam includes a main outlet works, an auxiliary outlet works, and an overflow spillway. The main outlet works are used to release water under normal operating conditions. These facilities include a submerged hexagonal intake structure leading to a 10-foot diameter concrete-lined circular tunnel. The tunnel leads to a gate chamber housing a 5-foot by 6-foot high-pressure control gate. From the gate chamber, a 76-inch diameter steel pipe leads to a control house with two 3.5 by 4-foot tandem gates and a stilling basin. A wye to a short 76-inch diameter steel pipe with a dished head was installed as a part of the original construction just upstream of the tandem gate control house. The hydropower facilities were connected to the existing main outlet works at this wye. The main outlet works has a capacity of 1,000 cubic feet per second (cfs).

The dam's auxiliary outlet works are used for releases during periodic shutdown of the main outlet works for maintenance. They are located at the right abutment of the dam and include a separate intake structure, a 72-inch diameter concrete-lined circular tunnel, directly beneath the centerline of the spillway. The auxiliary outlet works have a capacity of 600 cfs.

The Ruedi plant uses the BuRec-controlled outflows through the main outlet works to generate electricity. The flows are intercepted at the wye and diverted through a new penstock to a powerhouse located adjacent to the existing main outlet works stilling basin.

The electric power is connected to a 25 kV line in the local power grid in the Fryingpan River Valley and is delivered 15 miles west to a substation in Basalt, Colorado. The power is then wheeled by Holy Cross Electric Company to another substation owned by Aspen near its City limits. Aspen then uses the power for distribution to its own customers within its own electrical utility service area. The power generated by the project in effect replaces the power purchased by Aspen from other sources.

The project has operated as a baseload facility and does not have any diurnal peaking capacity. The flows diverted through the Ruedi plant are controlled by the BuRec. The Ruedi plant operating personnel are responsible for operating and maintaining the hydropower facilities in such a way that will not be detrimental to any of the existing project works.

Some uncertainty has existed regarding the future operations of Ruedi Reservoir largely due to three factors: (1) the Fry-Ark downstream senior conditional water rights have not yet reached their full levels, (2) downstream senior conditional water rights have not yet been perfected, (3) future development needs from downstream users such as the oil shale industry have not yet materialized, and (4) the BuRec has not finalized its marketing program

for the sale of water from Ruedi Reservoir. Each of these factors could alter the demands on the water in Ruedi Reservoir and, therefore, the patterns of water releases from the reservoir. To date, there has been little or no demand for the Ruedi water and the reservoir has, in effect, been operated primarily for recreational, wildlife enhancement, and flood-control purposes.

Aspen and Pitkin County, in cooperation with the BuRec, modeled the full range of possible future operating scenarios to determine the feasibility of the hydropower project. A wide variety of design configurations, including multiple turbines and a new afterbay reservoir, were considered to maximize the power production of the Ruedi plant. Because of the nature of the Fryingpan River and the intensive recreational use of Ruedi Reservoir and the Fryingpan River, the environmental effects of each alternative were carefully considered. It was concluded that the potential revenues which could be produced by the afterbay design were offset by the potential damages to the local aquatic resources. The afterbay design was dropped from further consideration.

Aspen and Pitkin County solicited competitive proposals from private industry for the final design, finance, construction and operation of the Ruedi plant. After a thorough review of 12 proposals, General Electric Company of Schenectady, New York, USA was selected to design, build and operate the facility under contract with Aspen and Pitkin County. Financing was provided by the Aspen electric utility using traditional revenue bonds. A single 5.0-megawatt Gilkes impulse turbine was installed, and the Ruedi plant was placed into operation in September 1985.

The project was completed within budget for a total capital cost of $4,100,000. The plant is now operated by General Electric under a full-maintenance and replacement contract for 20 years for a fixed annual cost adjusted annually by the Cost of Labor Index published by the Engineering News Record.

Permits and Project Schedule

A series of licenses and permits were required from several regulatory agencies. The Ruedi plant design and operating guidelines were sensitive to environmental issues. This approach made the permit process less contentious and time consuming.

The construction of the Ruedi plant required minor dredging and filling in the Fryingpan River. The project was designed such that all construction was limited to a 2-acre area between an existing road and the main outlet of the dam. The impacts of storm runoff and sedimentation during construction were limited by state-of-the-art water detention basins and sediment traps. The construction was scheduled to occur during low-stream flow periods so that

reservoir releases could be diverted through the auxiliary outlet works of Ruedi Dam.

The feasibility study for the project was completed in September 1982. The goal was set to complete the design, finance the project, obtain the necessary permits, complete the construction and place the plant into operation in three years. This was an ambitious goal considering the wide array of permits and the complexity of the construction required for the Ruedi plant.

The Ruedi plant was completed on schedule and placed into operation in September 1985. The achievement of such an ambitious goal is a tribute to the sound planning and cooperative spirit of the City of Aspen, Pitkin County, federal agencies, state agencies and private contractors who participated in the project. The following list provides an indication of the complexities of the issues involved in the permitting process.

1. Federal Energy Regulatory Commission
   Permit: License for Major Water Power Project
   Applied: February 24, 1983
   Received: October 1983
   Issues: Appropriate development and management of potential energy resource.

2. U.S. Forest Service
   Permit: Special Use Permit
   Applied: October 1983
   Received: 1984
   Issues: Environmental evaluation and review for consistency with forest plans. Issued finding of No Significant Impact.

3. U.S. Bureau of Reclamation
   Permit: Land Use License for Easement onto BuRec Lands
   Applied: August 2, 1983
   Received: 1985
   Issues: Proper development and safe management of Ruedi Dam and Reservoir.

4. U.S. Bureau of Reclamation
   Permit: Memorandum of Understanding for Construction, Operation and Maintenance
   Applied: 1982
   Received: 1985
   Issues: Safe, technically appropriate and legally permissible use of BuRec facilities.

5. U.S. Army Corps of Engineers
   Permit: Section 404 Dredge and Fill Permit
   Applied: 1983
   Received: 1984
   Issues: Dredging and filling waters of U.S. and potential impacts on in-stream and wetland resources.

6. Colorado Department of Health, Water Quality Control Division
   Permit: Section 401 Water Quality Certification
   Applied: January 28, 1983
   Received: March 31, 1983
   Issues: Potential effects on in-stream water quality and wildlife. Required special coordination of construction and operations activities with the Colorado Division of Wildlife.

7. Pitkin County
   Permit: Special Use Permit
   Applied: 1983
   Received: 1983
   Issues: Legally permissible and appropriate land use. Required several mitigation measures during construction and operations.

8. Eagle County
   Permit: Special Review
   Applied: 1983
   Received: 1983
   Issues: Plant site relative to county boundary uncertain. Full review and comments from neighboring county were fully considered and accommodated.

9. General Electric Corporation
   Contract: Final design, construction and operations and maintenance contracts
   Initiated: 1983
   Finalized: 1985
   Issues: Efficient, technically complete and competitively-priced proposal.

10. Holy Cross Electric and Colorado Ute Electrification Association
   Contract: Wheeling Agreement
   Initiated: 1983
   Finalized: 1985

Issues: Reliable and cost-effective delivery of power from Ruedi plant site to Aspen service area.

Economics

The City of Aspen distributes power to retail customers both inside and outside of its municipal boundaries. The Aspen service area consumes approximately 60,000,000 kwh per year of electrical energy. The Ruedi plant, after consideration for transmission line losses and wheeling charges, has provided 25 to 39 percent of this annual consumption, depending upon hydrologic conditions. Please refer to Figures 1 and 2 for the Ruedi production and Aspen consumption patterns.

Prior to the Ruedi plant, electric power was delivered to Aspen by the Public Service Company of Colorado (PSCo) under an "All Requirements Contract" at an average wholesale cost of $0.046 per kwh. In addition to supplying an important percentage of the Aspen electrical energy requirements from its own independent supply, the Ruedi plant provided Aspen the opportunity to diversify its sources of supply for the balance of its power needs.

One supplier, the Municipal Energy Agency of Nebraska (MEAN) has a customer base with a load pattern that peaks during the summer irrigation and air conditioning season. The peak loads in the Aspen service area occur in the winter months. These conditions presented the opportunity for MEAN to supply Aspen with supplemental power in a way which stabilizes the MEAN load pattern and allows MEAN to use its generation capacity in an optimum manner. Aspen and MEAN entered into a power supply agreement in 1985, and MEAN continues to supply power to Aspen. Aspen has further supplemented its supply and diversified its sources by purchasing power from the Western Area Power Administration (WAPA) at an economical rate.

Using this mix of supply sources, the cost of power for Aspen in 1992 (dry year conditions) was $0.041 per kwh or 11 percent less than the 1983 unit cost under its All Requirements Contract with PSCo. Under wet year conditions, the average cost would be $0.036 or 22 percent less than the 1983 cost because of better utilization of the Ruedi plant capacity. This produces savings in the cost of power ranging from $300,000 to $600,000 per year. When the capital costs of the Ruedi plant have been paid, the cost of energy will be further reduced to the range of $0.030 to $0.035 per kwh, creating a net benefit of $600,000 to $1,000,000 per year. These benefits are shared by Aspen, Pitkin County and the BuRec.

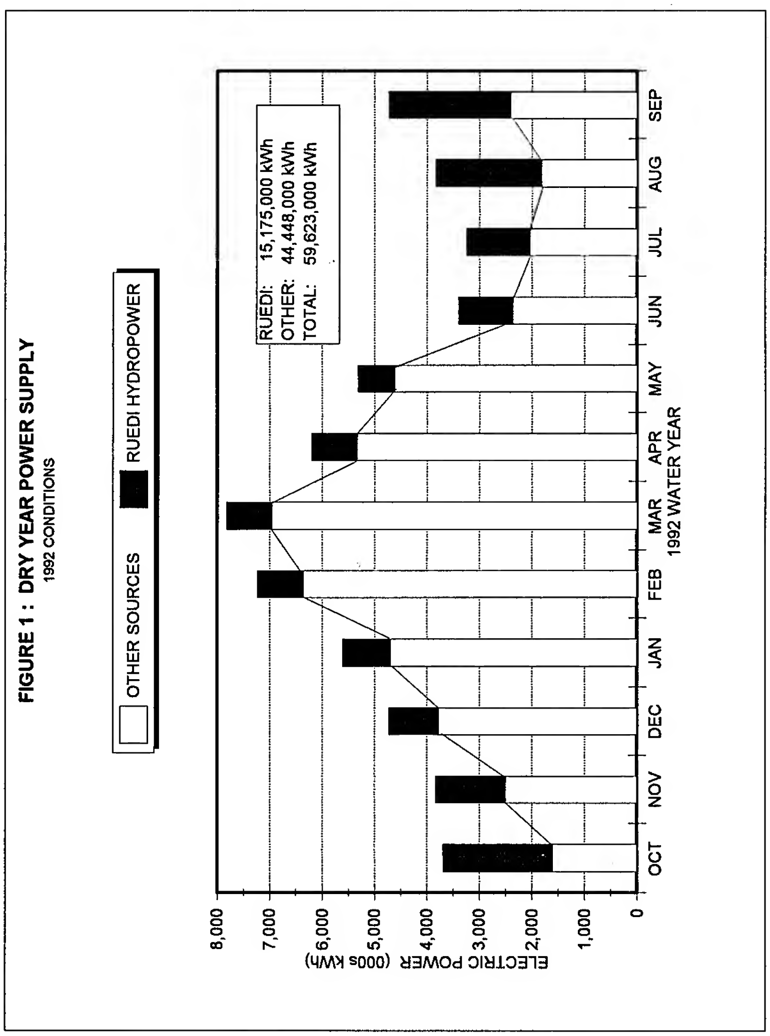

FIGURE 1 : DRY YEAR POWER SUPPLY
1992 CONDITIONS

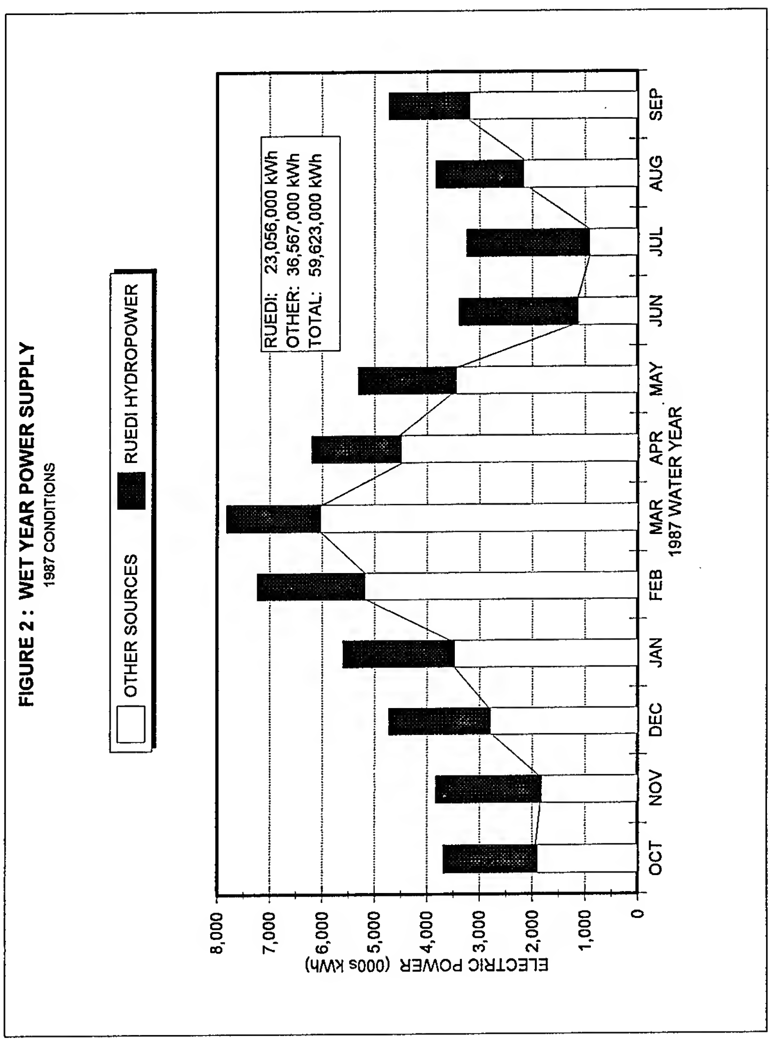
FIGURE 2 : WET YEAR POWER SUPPLY
1987 CONDITIONS
OTHER SOURCES
RUEDI HYDROPOWER
RUEDI: 23,056,000 kWh
OTHER: 36,567,000 kWh
TOTAL: 59,623,000 kWh
ELECTRIC POWER (000s kWh)
8,000
7,000
6,000
5,000
4,000
3,000
2,000
1,000
0
OCT
NOV
DEC
JAN
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEP
1987 WATER YEAR

MODERN CONTROL SYSTEMS FOR HIGH HEAD POWER PLANTS

Halvard Luraas [1]

Abstract

This paper deals with two power plants in Norway which would have been unstable according to traditional methods of calculation and design. In spite of this, however, extreme good stability is achieved by introducing new methods. The plants have extremely complex waterways, but they are very different and so are the methods applied to stabilize the plants. Jostedal power plant is stabilized by redesigning a small part of the waterway to a "damping shaft", while the Svartisen plant is furnished with an additional pressure feedback loop in the turbine governor. Concerning stability both methods applied alone or together may extend the established limits for the water inertia time of the waterway (The water starting time) T_w.

Introduction

In high head systems it has been common in Norway to introduce air cushions in order to stabilize the plants and to limit the pressure and speed rise. The alternative to air cushions has been surge shafts, but they tend to be long and therefore expensive and less effective than in low head schemes. In resent years other alternatives have been developed conserning stability. However, the optional solutions require more precise analyzing tools.

In Norway two high head plants, owned by the State Power Board, have been furnished with optional, but completely different solutions. In the Jostedal power plant parts of the waterway itself are designed in a special way to meet the stability requirement, but without surge shaft or air cushion. The Jostedal plant is today running with excellent stability combined with a high governor gain and normal unit acceleration time.

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[1]Senior Research Engineer, Hydro Power, Control Systems, Kværner Energy a.s, Kvaernerveien 10, P.O.Box 9277 Grønland, N-0134 Oslo, Norway

The Svartisen plant is furnished with an additional loop in the turbine governor (Kværner TC300) with feedback from the penstock pressure. Thus the solution does not require special measures in the waterway, but a careful analysis of the waterway-turbine system is necessary to design the governor and its parameter settings correctly.

The Svartisen Waterway System

The "Svartisen" ("The Black Ice") project in northern Norway, just north of the Arctic Circle near Bodø, consists of about 80 km of waterway tunnels. One of the more than 40 stream intakes is underneath permanent glacial ice. Five TBMs have been in operation, with four in use at the same time in one tunnel system. The project has several other unusual features, such as very large sandtraps and an elaborate energy dissipation arrangement in a supply tunnel, conf. ref [3]. The first of the two 350 MW units in the underground powerplant was commissioned early in 1993.

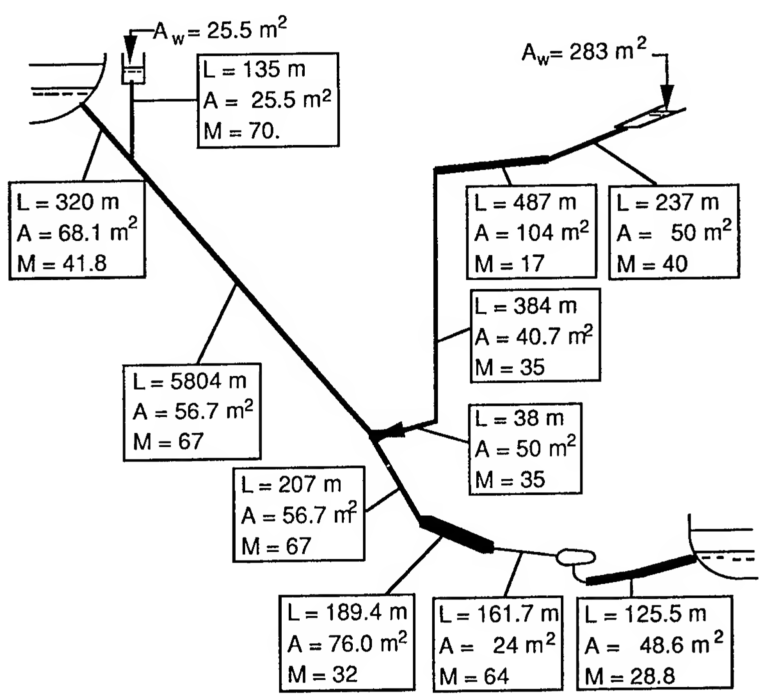

Fig. 1: Svartisen Power Plant: Simplified Schematic Waterway

Only a small part of the 80 km of waterway tunnel is of significance for the stability of the turbine governing, see fig. 1.

The key parameters for Svartisen at max load (two units) are:

P_{max}	=	2x386 MW	T_w	=	0.894 s
H_{emax}	=	580 m	h_w	=	0.297
Q_{max}	=	72.5 m^3/s	T_r	=	3.00 s
			T_a	=	6.37 s

According to these values the stability should not be a big problem. The water inertia time T_w (Water Starting Time) is less than 1.0 s. The low value for the water hammer constant, h_w, indicates though that we need a higher T_a/T_w-ratio than on low head plants, but still, with low governor gain, stability should be achieveable according to the rules of thumb.

When we dig into the analysis, however, the stability proves to be worse than expected. The reason is that we get a waterhammer surge between the two intakes. I.e. the difficult waterhammer does not arise from the ordinary reflection time, Tr = 3.0 s, but from a more seldom reflection between the two intakes. This surge corresponds to a reflection time of T_{r2} = 6.0 s. (Surge period 12 s) Thus it is located in the middle of the range of the crossing frequency which is a very bad location from a stability point of view. Note that the ordinary reflection time, T_r, corresponds to the second harmonic of T_{r2}. Therefore T_{r2} is easily excited by T_r. and they will both interact and "support" each other.

To stabilize the plant we may lower the governor gain below the normal range and/or increase the inertial mass of the generator. Then we have to accept greater frequency deviation. This is the price we will have to pay if we want to stabilize the plant in a traditional way, but without an air cushion in the waterway system.

The Svartisen plant has a very strategic location and will get a dominant position in its region. A dominant plant with a low quality frequency governing is not acceptable in Norway. Therefore the stability capability had to be improved.

The Alternatives

Due to the difficult topografy in the Svartisen terrain the surge shaft was ruled out. Thus, the original approach was to include a big excavated air cushion chamber. I. e. in spite of the fact that the instability at Svartisen is rather small it would mean considerable additional costs in the civil works to make the stability complete.

Different so called water column compensating methods have been suggested during the last two decades. Kvaerner has a good tool for simu-

lation of hydro power plants (waterway and turbine) in the frequency domain. This is a good starting point for looking closer into the possibilities for similar solutions. At Svartisen a pressure feedback from the penstock combined with the traditional PID-governor functions was developed in order to avoid the air cushion.

Stability Analysis

The "dotted" simulation in fig. 2 shows the frequency response of the open loop of the system without the pressure feedback. As we can see the transfer function crosses the negative real axis to the left of the singular point, (-1., j0.). Thus the system is unstable, and has a negative gain mar-

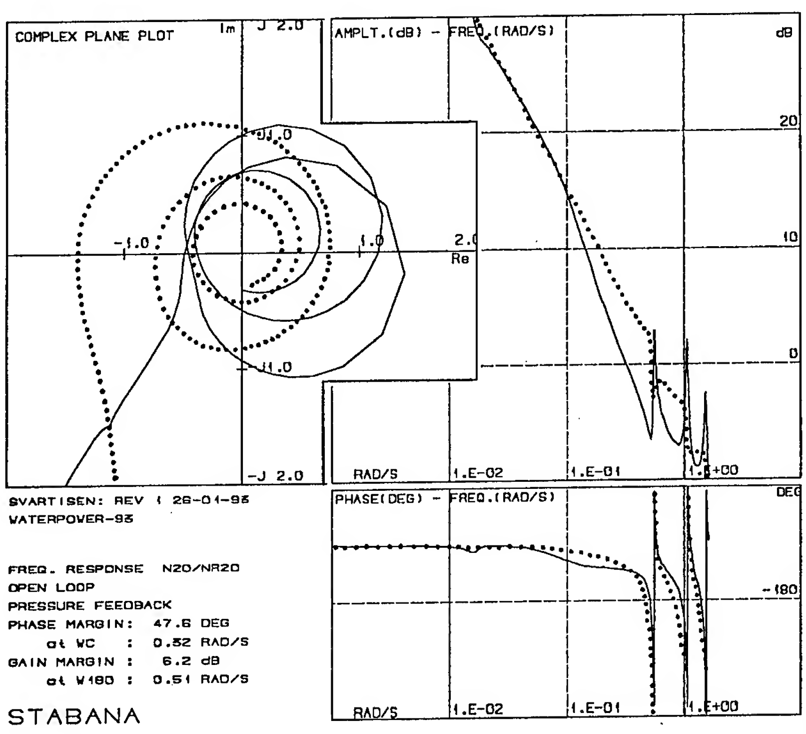

Fig. 2: Open Loop Simulation in the Frequency Domain without Pressure Feedback (Dotted Lines), and with Pressure Feedback (Solid Lines)

gin of -3.0 dB. If the system should be stabilized in the traditional way without air cushion, it would be necessary to lower the over all gain in the open loop. If we want to improve the gain margin from -3.0 dB to +3.0 dB it would be necessary to lower the gain in the turbine governor by a factor of 2. I.e. to increase b_t from 0.3 to 0.6. Alternatively the inertia of the generator could be increased by a factor of 2. These solutions imply that the frequency governing would be significantly worsened.

By the solid lines in fig. 2 we can see how the pressure feedback improves the stability in an obvious way. The open loop transfer function passes well to the right of the singular point. The gain margin is as big as 6.2 dB which is a very good margin for power plants. From the figure we can see that the phase margin is even bigger (47.6 deg.).

Fig. 3 is a closed loop simulation of the system with pressure feedback. The transfer function is speed versus load disturbance. The maximum speed deviation is -3.5 dB, i.e. 0.67. This value says that a one percent sinusoidal steady state load disturbance will give a maximum of 0.67% speed deviation around the crossing frequency, i. e. well below 0 dB which is often used as a criteria for closed loop. The conclusion is that the stability criteria is more than fullfilled both in open and closed loop.

Pressure Feedback

By comparing the amplitudes of the open loop we find that by means of the pressure feedback we have lowered the gain around the crossing frequency significantly, but for low frequencies the gain is still the same as before. If this should be done by a serial filter in the ordinary PID-loop it would destroy the phase margin completely. Therefore we need a parallell loop with less phase problem to be able to perform the necessary correction around the crossing frequency without loosing the stability.

From the simulation we can see that we do loose some of the phase, but not a 90 deg. step as we will easily loose in the ordinary PID-loop if we try to lower the gain only around the crossing frequency by a serial filter.

Based on this it seems probable that a feedback from the flow could give us even better phase qualities, but to measure the pressure in the penstock is a simple task while the flow is more difficult to achieve. Also, in order to avoid that the additional feedback loop shall introduce an unintended permanent speed droop, we must add a derivative multiplier to the feedback loop. I. e. the flow signal must be differetiated twice.

By the pressure feedback it is possible to maintain the gain for frequencies below the crossing frequency and simultaneously lower the gain around the crossing frequency. Thus the pressure feedback makes it possible to keep both the control qualities and the stability capability.

The pressure feedback tells the governor to be careful in the dangerous part of the frequency range where we find the water hammer surges. It tells the governor not to excite the waterway in this range because the waterway is rather "irritable" at these frequencies. In the rest of the range for the turbine governor it may behave in an ordinary way and with a fully acceptable gain.

Brief Description of the Governor

The control algorithms are implemented on the Kværner Turbine Controller hardware which is based on an ordinary industrial PC. Thus it is easy to modify the standardized algorithms to fit special demands. The Svartisen governor, named Kværner TC300, is an extended version of the Kvaerner Turbin Controller TC200. In TC300 the pressure feedback algorithm is included.

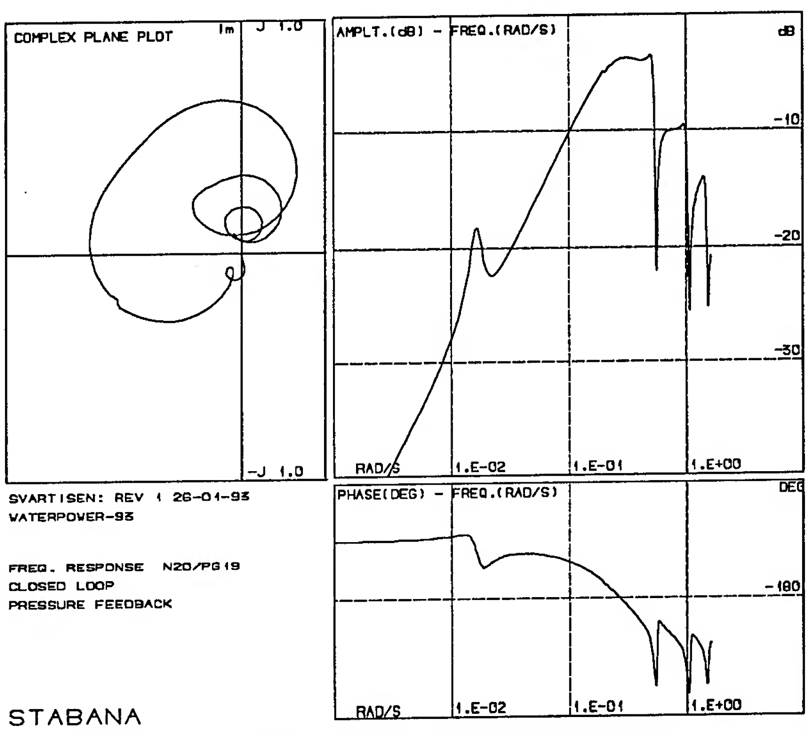

Fig. 3: Closed Loop Simulation with Pressure Feedback.

The Jostedal Waterway System

The Jostedal Power Plant (5 jet vertical Pelton commissioned late in 1989) in western Norway does also constitute a complex waterway system with 19 intakes from stream and rivers and about 35 km of tunnels. In order to obtain stable governing at full load on isolated load the first approach was to excavate a surge shaft. But due to the high head the surge shaft had to be about 800 m long and thus expensive. Therefore the client wanted to look at the possibility to design the first stream intake (seen from the turbine) as a surge shaft. The length, however, from the turbine to this intake is about 5200 m long so an ordinary surge shaft will fail. The waterhammer surge will appear below the crossing frequency and for reasonable governor gain it will destroy the phase angle completely.

The caracteristic parameters for the Jostedal plant at rated load are:

P_r	=	288 MW	T_w	=	0.97 - 1.40 s
H_r	=	1130 m	h_w	=	0.11 - 0.084
Q_r	=	28.5 m3/s	T_r	=	8.6 - 16.8 s
			Ta	=	5.94 s

Due to the special solution concerning stability the parameters for the waterway are difficult to determine precisely. If the values are determined from the turbine to the first waterlevel, it means that we are going through the damping shaft. The extreme friction in this shaft means that the rest of the waterway also will contribute to the determination of T_w and h_w. The reflection from the damping shaft is not unique. I. e. the reflection time is also difficult to determine in a strict way.

Stability Analysis of Jostedal Power Plant

The analysis, which is described in more detail in ref. [4 & 5] shows that instead of an ordinary surge shaft a "damping shaft" with extreeme damping will solve the stability problem. A small stream intake is used as damping shaft. In the branching to this intake a 10 m long concrete plug with a narrow opening (0.3 m^2) is used to achieve the damping. The cross section of the branching is 50 m^2 and the main tunnel is about 34 m^2.

Speed and pressure rise are a minor problem because the plant is furnished with a Pelton turbine. The problem with standing waterhammer surges with small damping after load rejection will be significantly reduced in systems with damping shafts. Simply due to the fact that we do our best to destroy the surging systems by introducing damping shafts.

What is a "Damping Shaft"?

The Jostedal stability is based on a completely different concept than the pressure feedback used at Svartisen. In the Jostedal waterway we

have introduced a "damping shaft". The friction in the damping shaft is so strong that no waterhammer surge going up this shaft can exist. At the same time it is open enough to drain the waterhammer surges that tend to go further up in the waterway. The balancing of these two phenomena results in a "window" where stability can be achieved.

The draining effect of the surges that goes further up in the waterway is due to the fact that pressure oscillations makes water go in and out of the damping shaft. Thus the energy is sucked out of the surges. There is one exception, however. If the surges have a node in the branching point of the damping shaft there will be no pressure oscillations in this point and the draining effect will not work. Therefore the positioning of the damping shaft cannot be quite occasional. Conf. fig. 5 in ref. [5]. In spite of this the range for the location of a damping shaft is much wider than for an ordinary surge shaft.

It should be noted that in spite of the strong damping in the damping shaft the headlosses in the main headrace of the waterway is not affected.

On Site Tests at Jostedal Plant

Measurements at site of frequency response of penstock pressure as function of needle stroke show good agreement with corresponding simulations so the mathematical modeling of the waterways has been fairly good. Therefore the open and closed loop simulations should also be correct, conf. ref. [2]

The Jostedal plant is also measured during operation on isolated load. This was achieved by means of an integrated aggregate simulator simulating an isolated grid, but using the physical waterway and turbine. In this way it was possible to verify that the plant is stable on isolated load.

In Jostedalen the original instability was worse than at Svartisen, but in spite of that it is today operating with a gain equal to 3.3, i. e. $b_t = 0.3$, and the gain margin is about 6 dB at full load and a one percent load disturbance. The stability margins will increase as the load disturbance increases because the damping shaft is a dominant nonlinear element.

Conclusions

The stability problem has been solved quite differently for the Jostedal and Svartisen hydro power plants. However, they have one thing in common, namely the tool for analysing the stability. The theory , ref. [1], and the program were developed in parallell at Kvaerner. The program called Stabana is refined from time to time, but the main theory for simulating waterways and turbines in the frequency domain has been quite established in the last decade. It is verified by several frequency response tests in full scale. This theory, lately referred to as the "Structure Matrix Method",

cf. ref. 2], is the basis that makes it possible to investigate complex waterway scheemes accurately together with new algorithms and thus find new control strategies and solutions. In the Jostedal case Stabana was used to redesign parts of the waterway to obtain stability while a new control algorithm proved to be a good solution for Svartisen. This illustrates that the waterway design can be an important concern for the turbine governor and vice versa.

Systems with surge shafts result in surging systems, but we make the systems such that the resonance frequencies are above the crossing frequency. That is why the solutions are stable which means that the distance from the turbine to the first waterlevel must be below a certain limit. By damping shafts we are doing the oposite. Then we do what we possibly can do in order to destroy the surging systems. Thus the solution with damping shaft is really the most natural solution concerning stability. Earlier, however, we did not have the possibility to analyse solutions like that. Till now we have preferred to deal with clean surging systems where the frequencies, T_r, T_w and h_w can easily be predetermined.

With the analyzing methods available today the possibilities are here. One of the advantages with a damping shaft is that it does not need to be as close to the turbine as a surge shaft. The civil work will usually be less expensive. In the Jostedal case the branching part of a small stream intake was redesigned, i. e. the additional costs were just a minimum. However, the analyses will be far more extensive and they need to be more precise.

The two methods demonstrated by the Jostedal and Svartisen plants may very well complement each other. By the pressure feedback it is possible to lower the gain around the crossing frequency. Thus the crossing frequency will also drop slightly, but the gain in the normal range for the turbine governor is as before, i. e. according to normal criteria.

The pressure feed back makes it possible to extend the limits for the penstock. I. e. the limits for an acceptable T_w concerning stability may be extended if we may apply the pressure feedback.

By a damping shaft waterhammer surges below the crossing frequency may also be handled, and the T_w-limits concerning stability may be further increased in some cases if the criteria for speed and pressure rise during load rejection is met. It should be noted, however, that in order to decide wheather the methods will work or not, careful analysis is necessary.

As a result of the surging systems we get when we are using surge shafts, we may easily get standing waterhammer surges after load rejection. This problem will be significantly reduced by damping shafts .

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THE SVARTISEN HIGH HEAD PROJECT,
TURBINE DESIGN AND MANUFACTURING

K. Bratsberg[1]

SUMMARY

The high head Francis turbines for Svartisen powerplant were designed using an advanced CAD system based on parametric design. The system stores all information needed for the manufacturing in one project data base. The system includes utilities for information conversion to general CAD drawing systems, NC machining, FEM structure analysis and flow simulation. The design information in the project data base was used for a number of advanced analyses and to build a model turbine. Finally the turbine was manufactured using the same database.

INTRODUCTION

The Svartisen hydro power plant is situated just North of the Arctic circle in the northern part of Norway. Since it is close to the Svartisen glacier, Norway's second largest glacier, runoff from the glacier represents a major water source for this plant. The Svartisen hydro scheme is developed and will be operated by the Norwegian State Power Board.

The power plant will have two Francis turbines when completed. The first turbine will be commissioned in early 1993. The turbines have the following specifications:

Head, rated .... 543m (1781ft) min/max 500/580m
Speed, rated 333rpm
Output, rated 350MW

[1] Project Manager, Kvaerner Hydro Power, Inc. 100 First Stamford Place, Stamford, CT 06902

The combination of high head and large output makes these turbines special, as they have the world's largest output for Francis turbines with head above 500m (1640ft).

Figure 1 shows a vertical section of the underground (cavern) power station with generator and turbine, and a cross section of the turbine.

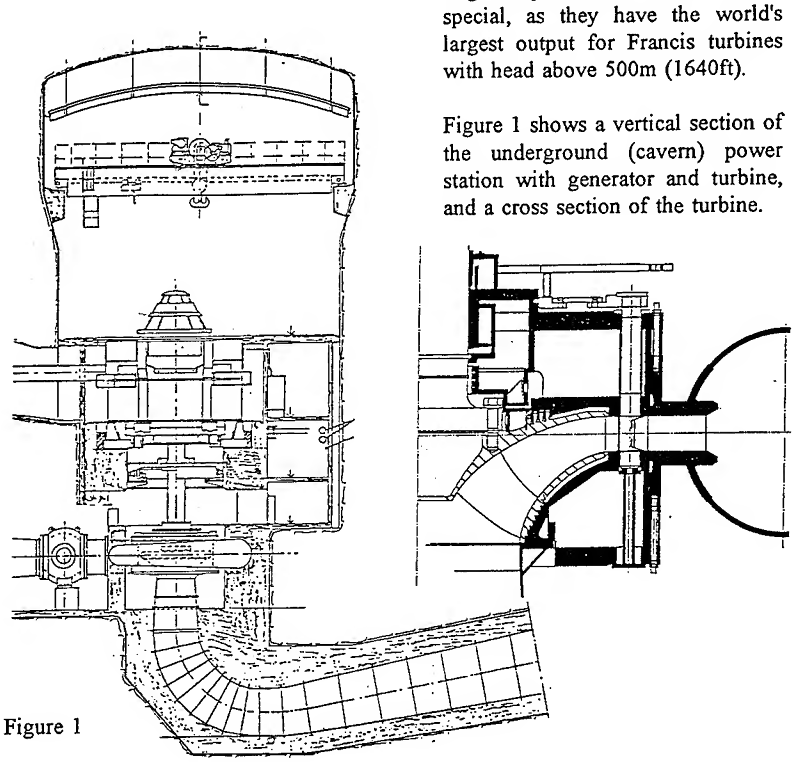

Figure 1

DESIGN

The turbines for Svartisen have been designed using the KVAERNER CAD system with parametric design, which has been operating and continuously improved during the last 12 years. The last version of this system allows a complete assembly drawing to be made in a few hours, with simplified stress and deformation analysis carried out where needed, figure 2. The system is developed by KVÆRNER and uses the SINTEF developed information management system FIDA, Stenberg [1984].

A design database contains all the information required to design a turbine. The information is represented by design rules, parameter tables, and special representations of complex geometries. The database includes optional designs for many parts, e.g. four different cover designs, numerous guide vane profiles and runner vane geometries for different specific speeds.

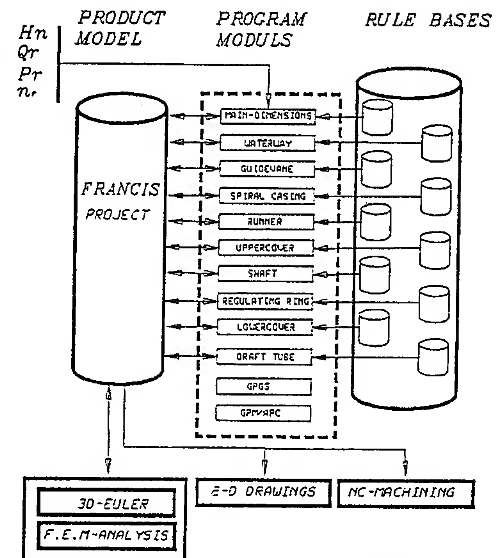

Figure 2

The design process starts with selection of turbine speed and main dimensions, including runner series, based on head and output/flow, incorporating constraints with respect to submersion, etc. The designer then runs a set of programs that use the design database with its rules and data to build up a complete data set in physical coordinates for the specific project. Figure 2 shows the different modules and their sequence from top down. The turbine parts are designed around the geometry of the wet surfaces.

The system gives a proposed design in each module which can be readily evaluated as the system incorporates interactive graphics displaying the design in all the important views. The user can choose among options and may give geometric input to modify the proposed design. The changes will instantaneously show on the computer screen. Simple stress and strain (deflection) calculations are also provided where needed, so the designer can optimize the design with respect to positioning of bolts, plate thicknesses and so on. Plates and bolts are picked from standard tables.

The stay ring design is a good example. When the spiral casing geometry is determined, the system proposes a stayring design. A cross section of the stay ring is displayed and the stress distribution is plotted over the stay vane. The user can then change parameters like thickness, radial position and the length of the stay vane to optimize the stress distribution in an interactive process.

For advanced structure- and flow- analysis of different turbine parts, like three dimensional stress analysis of a turbine cover, the geometry in question can be extracted and converted by special interface software to the right input format for the desired analysis.

Drawings can be plotted directly from the design programs during the design process for review purposes, with main dimensions put on by the system. After design completion the geometry is converted for input to standard CAD software, like Autocad, for preparation of shop drawings. General CAD software is better suited

for interactive drawing when completing annotation and details for the finished 2-dimensional shop drawings.

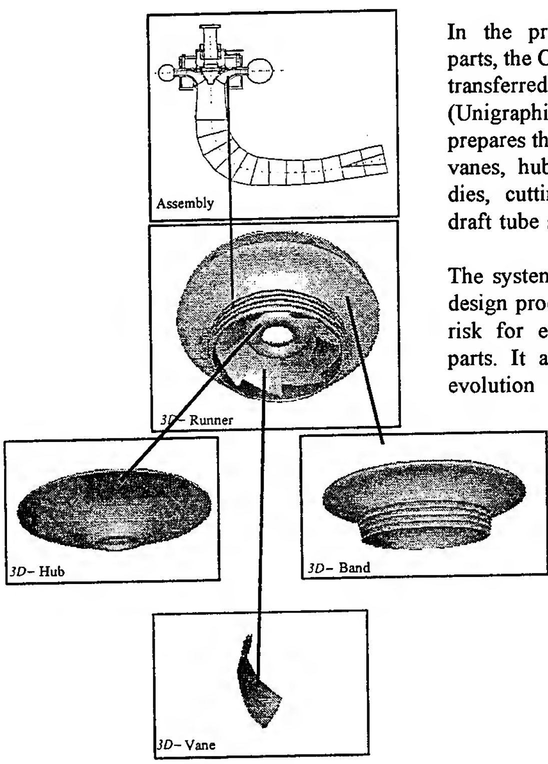

Figure 3

In the production of the turbine parts, the CAD system information is transferred to the CAM system (Unigraphics). The CAM system prepares the NC machining of runner vanes, hub and band, runner vane dies, cutting of spiral casing and draft tube shell plates, etc.

The system saves much time in the design process, and it minimizes the risk for errors and misfits of the parts. It also structures the design evolution as design improvements are directed towards the parametric database and not the specific project.

Figure 3 shows a visualization of a runner and its different components.

FINITE ELEMENT ANALYSIS OF STAYRING

For this analysis, the geometry and boundary condition data was transferred from the project data base to the pre-processor by an interface program. As volume information was preserved, the pre-processor could build the grid directly and automatically. The analysis could then be carried out without manual input.

Figure 4 shows a post-processor result with the grid in two views and displacement and stress results. The analyzed model has 242 elements and 5,299 degrees of freedom. The commercial software used for the analysis was FAM/FEGS Ltd., SESTRA, and POSTFEM from VERITEC.

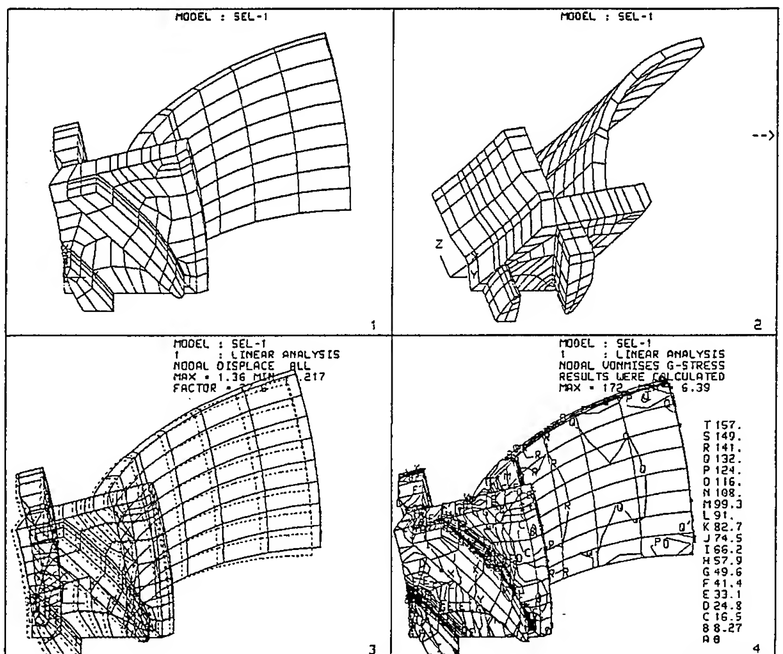

Figure 4

ANALYSIS OF RUNNER AND GUIDE VANE FLOW

Runner: The runner flow was analyzed using a 3 Dimensional Euler (inviscid) solver for incompressible flows described by Billdal et al. [1989]. This is a finite-volume formulation with Centrifugal and Coriolis terms included in the momentum equations, in order to simulate flows in rotating systems.

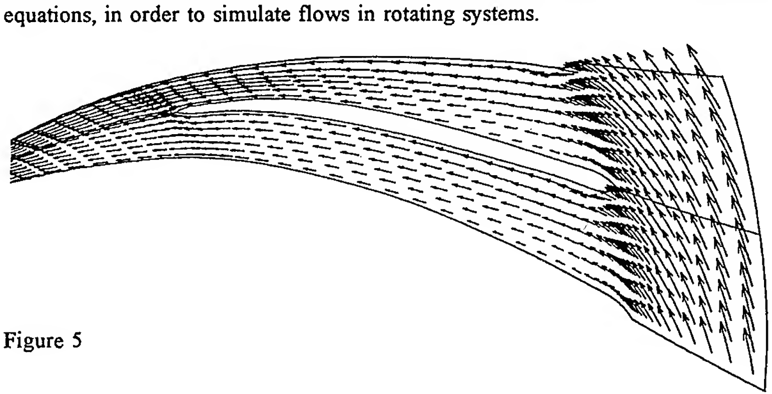

Figure 5

The simulation was complicated by the splitter blades that were used for this runner design, figure 5. The grid had 16,080 nodes, and 4,000 iterations were necessary to reach steady state. The flow-pattern was found to be satisfactory with good inflow conditions. The simulations showed the positive effect of the splitter vanes, and no changes were made to the model turbine based on this analysis.

Guide vanes: This analysis was carried out by Chen [1992] as part of a separate thesis work (ref. model tests). The flow through the guide vane cascade was simulated by a similar program to the one used for the runner, a finite-volume formulation. The analysis was 3-Dimensional and included viscous effects and turbulence (Baldwin-Lomax turbulence model).

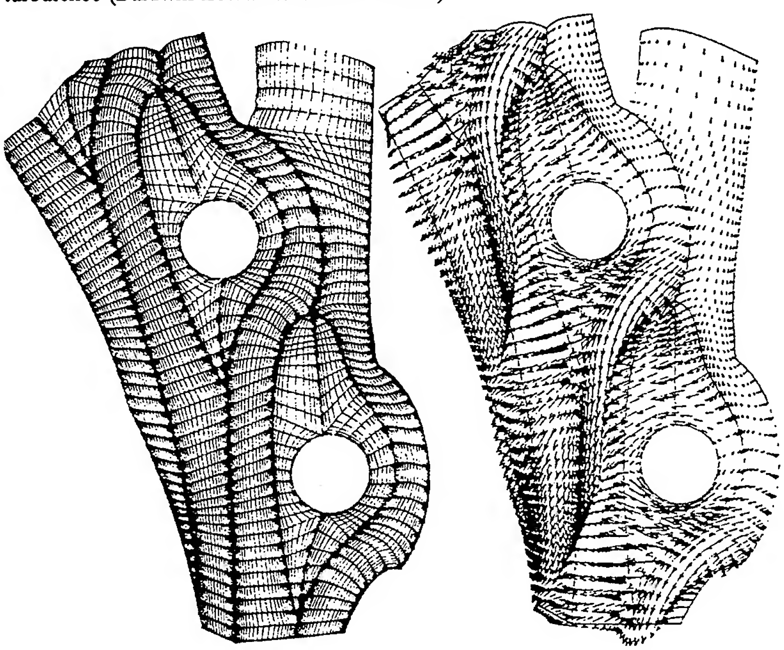

Figure 6

Figure 6 to the left shows the grid used in a view perpendicular to the axis of rotation through the gap between vane and turbine cover. Velocity vectors are shown to the right in the same view. The shaft transition gives a local asymmetry at the vane end faces. The computational results were in good agreement with the model test results (ref. page 9), showing how the leakage increased the vortex formation in the guide vane flow. The simulation comprised the stay and guide vane channel, and the gap between vane and cover in one single model. The gap was varied between 0.13% and 0.5% of the width of the channel.

RUNNER PRODUCTION

The runner for Svartisen is a welded design, manufactured entirely of high-strength martensitic stainless chromium-nickel steels.

The vanes are made from steel plate and hot-formed in dies. The hub and band are made from castings.

The inner contours of the hub and band are machined before assembly and welding.

The vanes are NC machined to correct shape, including machining of the welding grooves, figure 7. The project data base contains the information needed for this process, including mapped vane surface and mapped welding grooves.

The dies are NC machined to give the exact shape of the finished vane. Figure 8 shows a CAD model of a die with 3D spline representation of the surface. The tool paths drawn show the principle of machining.

Due to the deformation of the vane under pressing the vane surface maps, used to machine the vane, must be compensated in accordance with experience and computations.

The pressing is shown in figure 9. The vane is heated and then


Figure 7, Machining of vane

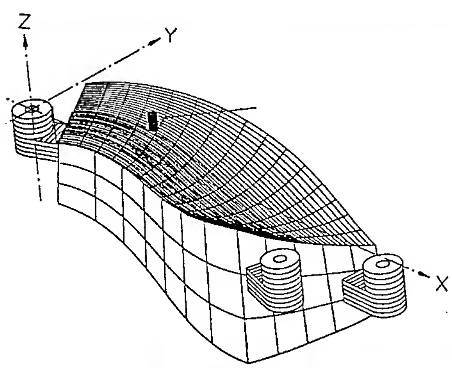

Figure 8, Cad model of die

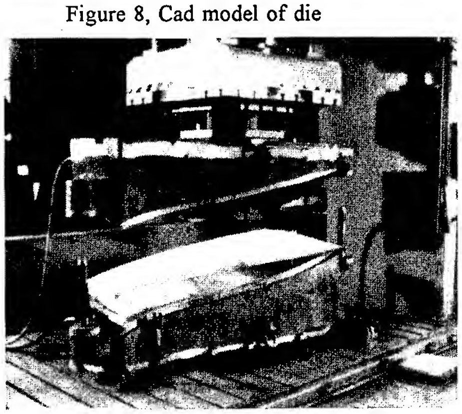
Figure 9, Pressing of vane

pressed to the correct shape. Much effort has been put into optimization of the forming process, especially in the handling of the hot vane to minimize the cooling effect and to speed up the process.

The vanes fit well into the hub and band after pressing, and the assembly can be carried out very efficiently.

Figure 10 shows the runner with vanes ready for welding. The splitter vanes are not mounted at this stage.

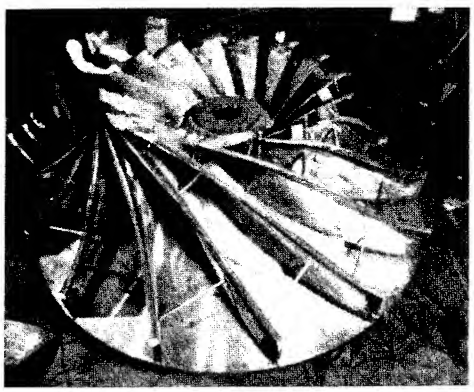

Figure 10, Runner assembly

MODEL TESTS

In addition to the regular efficiency and cavitation tests a number of other investigations were carried out on the model turbine for Svartisen. These investigations were both aimed at optimizing the design and improving the general understanding of the processes in a high head Francis turbine:

- Comparison of two different draft tube geometries.
- Optimization of stayvane and guide vane positioning.
- Measurements of transient pressures in the guide vane cascade.
- Transient pressure measurements inside the runner.
- Investigations of leakage flow over guide vanes.

Draft tube: Comparison was made of two draft tubes with approximately same distribution of through-flow area, but with different cross sections. One draft tube had circular cross sections, the other had elliptic-rectangular cross sections. In all other respects, the basic geometric dimensions were nearly the same. The circular geometry has practical advantages with respect to strength and air escape when concreting.

There was no significant difference found in the measured model turbine characteristics within the normal region of operation resulting from the type of draft tube used.

Stay/guide vane: The covers with guide vanes were rotated in small increments relative to the stayring. As the number of guide vanes is the same as the number of stayvanes, the positioning is periodic with 360/Z. A considerable difference in efficiency was measured depending on the relative positioning. The optimum was found when the guide vanes and stayvanes were staggered relative to the path of flow. Placing stayvanes and guidevanes in line gave the highest losses.

Guide vane leakage: As part of a separate thesis work by Chen [1992], a special test rig was built containing a half-size model of the last quarter of the spiral casing, figure 11. The model included stayvanes and guide vanes with a transparent stay and guide ring to enable observation. The leakage flow was observed by injection of dye and the flow field between the vanes was measured with Laser-Doppler Velocimetry for different end-face clearances.

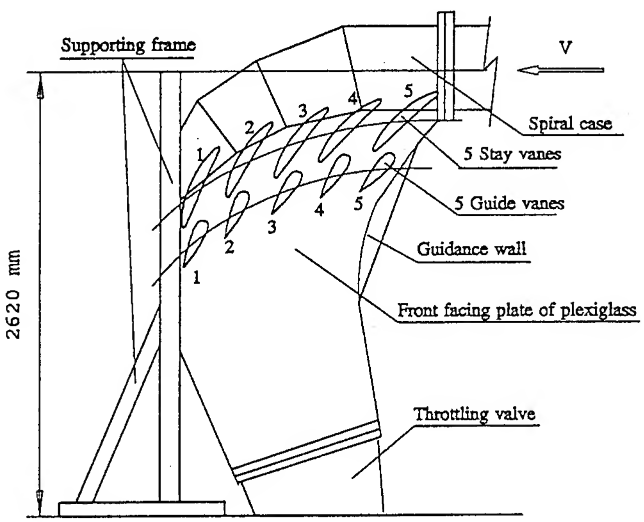

Figure 11

Understanding of the mechanics of the guide vane leakage is important because it has proved to be the largest single source of loss in high head Francis turbines and it is usually increased with turbine wear. The secondary loss caused by low energy leakage water disturbing the runner inflow can be larger than the direct head loss through the guide vanes. The investigations provided important information for further computer analysis.

CONCLUSION

Sophisticated computer systems have great advantages in turbine design and manufacturing. It becomes a powerful tool especially when incorporating customized routines for automatic design. The saving in cost and time, when a system is established, is obvious. In addition the importance of having only one set of information that describes the design is important.

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ENGINEERING MT. HOPE
--A STATE-OF-THE-ART PUMPED STORAGE PLANT--

Paul F. Shiers[1] and Frank S. Fisher[2]

ABSTRACT

The Mt. Hope Waterpower Project, a 2000 MW, six-unit underground pumped storage development, received its license from the Federal Energy Regulatory Commission (FERC) in August 1992. The development phase of the Project has continued with the recent completion of a Project Definition Study which updates the Project's conceptual design, cost estimate, and construction schedule. Although, the Project is located close to the metropolitan New York Load Center, when construction is complete, the power generating facilities will be essentially invisible.

This paper describes the main Project conceptual design features which have been developed during the recently completed Project Definition Study and the challenges which the project designers must overcome. For example, the upper reservoir has to be quarried out of rock at the ground surface, the lower reservoir has to be excavated like a mine 2500 feet (762 m) below ground surface, and the power generation facility will be enclosed in caverns 2800 feet (854 m) below ground surface.

Through the years, the philosophy of pumped storage has evolved from simple energy storage and peaking capacity to a much more comprehensive system management tool. Mt. Hope builds on past

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[1] Vice President, Stone & Webster Environmental Technology & Services, 245 Summer St., Boston, MA 02210 - USA
[2] President, Mt. Hope Hydro, Inc., 627 Mt. Hope Road, Wharton, NJ 07885 - USA

achievements within the pumped storage industry and adapts this progress to provide a unique facility capable of offering system-wide benefits to the regional grid. With the lower reservoir almost directly below the upper reservoir, the ratio of water conduit length to available head is almost equal to the ideal value of unity. Furthermore, the available head has been selected to achieve the highest head that the pumped-turbine designers consider reasonable for state-of-the-art reversible pump-turbines. High head and short conduits also optimize fast-response capability which provides system regulation benefits not readily available from other types of peaking power plants.

INTRODUCTION

The Mt. Hope Waterpower Project is the third-largest hydro project that the FERC has ever approved and the largest project licensed by the FERC in 15 years. Mt. Hope will also be the second-largest pumped storage plant in the world and will use the highest head reversible single stage pump-turbines in the world.

These state-of-the-art pump-turbines will be provided by Kvaerner a.s, of Norway one of the world's principal suppliers of water turbines and a major backer of the Project. The generator-motors will be provided by ABB Generation AB of Sweden, one of the world's major suppliers of electrical equipment. The engineering team is being led by Stone & Webster Engineering Corporation of Boston, MA, a major engineer-constructor of hydroelectric and pumped storage power plants in the USA. The project team also includes the engineering firms of Golder Associates and TAMS Consultants.

The Mt. Hope Waterpower Project is intended to provide stored response capability for use as a utility system tool. The primary importance of this type of plant is its ability to respond rapidly to continuously changing conditions and thus maintain system reliability. The plant will also help the system recover quickly following any system disruption. Emphasis is placed on maximizing economy of operation -- for the interconnected plants in a utility system as well as for the pumped storage plant -- and benign environmental coexistence. The plant will, of course, provide additional system capacity, but this has been considered to be of secondary importance compared to the primary objective of providing a flexible system management tool. The decision to tailor the Project toward one purpose rather than another affects the nature of the choices to be made in the design process.

Thus, in addition to the normal pumped storage functions, Mt. Hope will provide dynamic and fully dispatchable response capabilities. These

capabilities will increase system reliability and reduce inefficient or life-shortening cyclical use of thermal stations. The plant will also provide the utilities with a greater diversity of fuel supply.

The Mt. Hope Waterpower Project will offer the following features to assist the utilities in meeting their system objectives:

- Spinning reserve and the prompt availability of additional capacity at essentially any time on very short notice.
- System load following capability which can reduce unit commitment and cycling of thermal generating plants in the system.
- System frequency regulation.
- Power factor and voltage regulation capability.
- Improved use of the regional transmission system.
- Storage of low-priced off-peak energy.
- Transfer of low-priced off-peak energy from clean air attainment areas to non-attainment areas (the stored energy can displace thermal energy sources which contribute to air pollution in non-attainment areas).

The final design of the plant will be tailored to the needs of the utilities. For example, options such as frequent and rapid mode change capability will have a significant effect on the final design. The present conceptual design calls for 20 mode changes per day, power generation in less than 1 minute from standstill, and 80 percent of full load in 10 seconds from the spinning in air mode.

GENERAL FEATURES OF THE PROJECT

The Mt. Hope Waterpower Project is located at Mt. Hope in Rockaway Township, Morris County, New Jersey approximately 2 miles (3.2 km) northeast of the town of Dover, New Jersey and 35 miles (56 km) west of New York City. Being near a major population center is both an opportunity and a challenge. The opportunity is to place a major generating facility near a load center. The challenge is to do this unobtrusively and in harmony with the surroundings of a heavily populated, but environmentally sensitive, suburban region. The opportunity will be realized and the challenge will be met by transforming an existing industrial complex into a

site that is compatible with its suburban surroundings.

The project site includes the inactive New Leonard Mine Complex, an active rock quarry, and the existing Mt. Hope Lake. The Project will be arranged to avoid the existing mine workings, minimize the effect on future activities of the quarry, and contribute to the quality of the surrounding neighborhood. Areas which had been industrial eyesores will be transformed into parks, recreation areas, and restored historic buildings which will belie the mighty power complex hidden from view below.

All pumped storage projects must have two reservoirs at different levels. Power is generated into the utility grid whenever it is needed by discharging water from the upper reservoir through the pump-turbines to the lower reservoir. At the earliest opportunity, this water is pumped back by reversing the rotation of the pump-turbines using energy from the same grid. This results in a net consumption of electricity but benefits a utility system by improving its overall performance and by leveling the system demand.

At Mt. Hope, the water used for this process will be contained in a closed loop which will not be connected to outside water bodies. There will be an upper reservoir excavated in a small plateau, a lower reservoir excavated from rock 2500 feet (762 m) below ground surface, a powerhouse complex constructed 2800 feet (854 m) below ground surface, shafts connecting the underground complex to the surface, and a transmission line connecting the plant to the utility grid. The overall arrangement is shown in Figures 1 and 2.

UPPER RESERVOIR

The upper reservoir site is tightly bounded by Picatinny Arsenal property on the north and west, abandoned mine workings on the southeast, recently quarried low ground on the northeast, Mt. Hope Road to the south and the Mt. Hope Fault to the north. The inactive Mt. Hope Fault is not exposed at ground surface, but it is believed to generally follow the northwest trending topographic low area north of the reservoir.

The upper reservoir will have a surface area of approximately 55 acres (22.3 hectares) and an active storage volume of 5000 acre-feet (6.2 million cubic meters). The total water level fluctuation will be about 110 feet. Water for the initial filling of the upper reservoir was originally anticipated to be obtained from Mt. Hope Lake. However, water from the presently flooded New Leonard Mine complex can supply approximately half of the reservoir capacity, with the remaining water coming from Mt. Hope Lake and mine inflow. Construction of the upper reservoir will take place

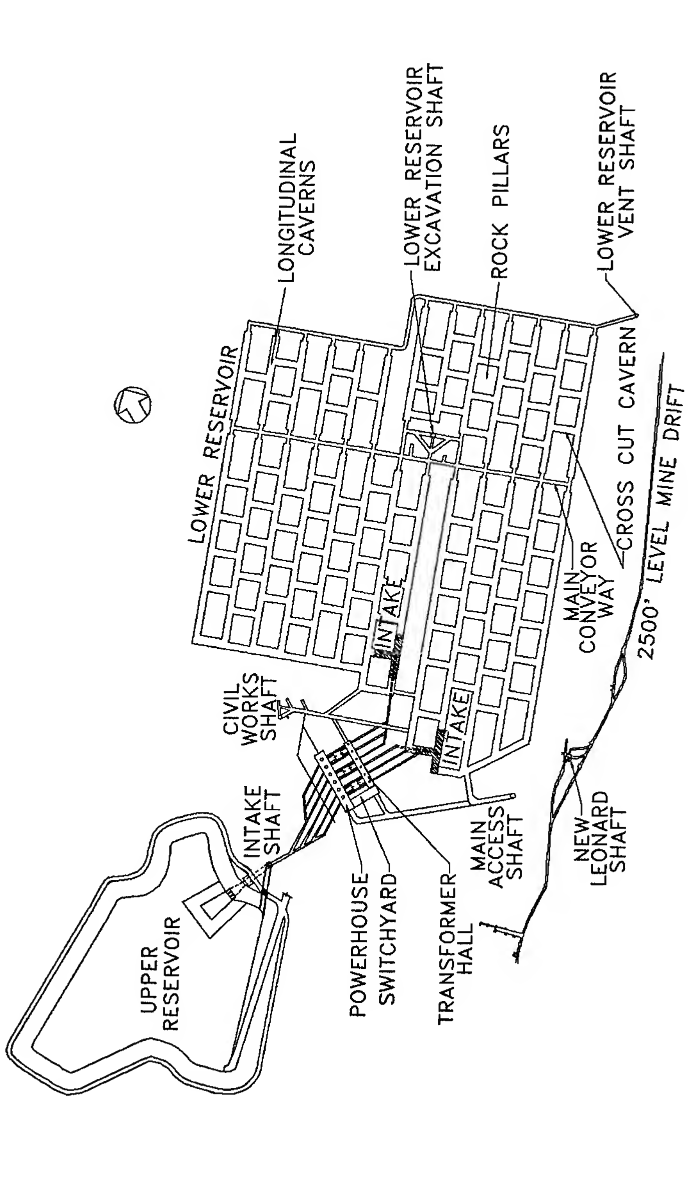

FIGURE 1 Mt. Hope Waterpower Project - Plan

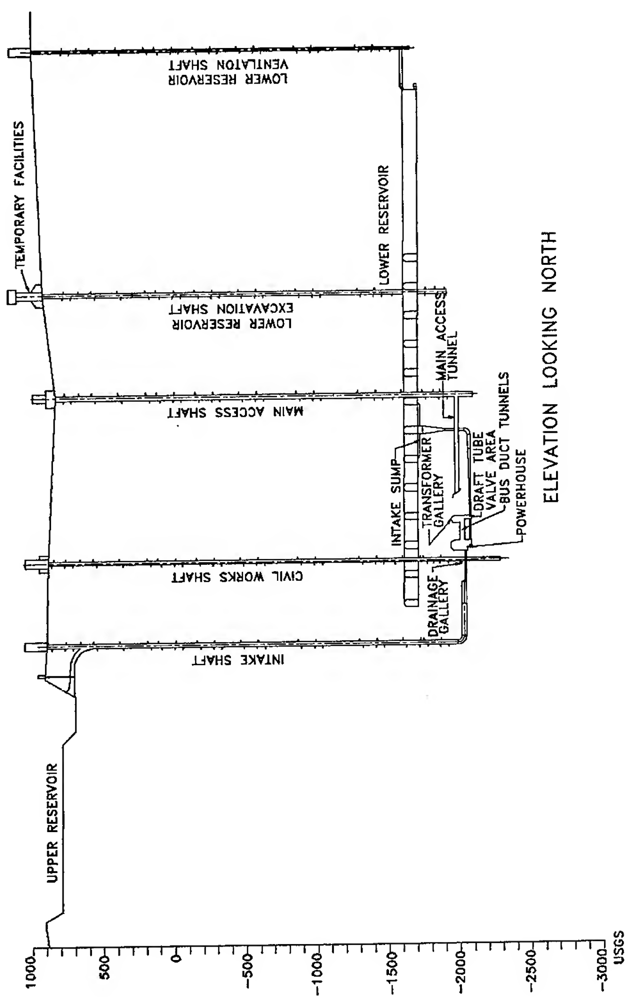

FIGURE 2 Mt. Hope Waterpower Project - Cross Section

early in the construction program. This will allow early transfer of water from the mine followed by pumping from the mine and Mt. Hope Lake exclusively during periods of relatively high inflow, over a 3-year period.

Because the system is essentially a closed-loop system, there will be very little need for long-term make up water. In most years, rainfall will offset expected evaporation.

LOWER RESERVOIR

The lower reservoir will be excavated like a mine 2500 feet (762 m) below the ground surface and will have an active water storage volume of 5000 acre-feet (6.2 million cubic meters), the same as for the upper reservoir.

Constraints in locating and orienting the lower reservoir are the result of plant performance requirements, property boundaries, geotechnical considerations, and quarry operations. The major challenges that were faced included:

- Selecting an elevation which allows the maximum acceptable pumping head.
- Positioning the reservoir close to the powerhouse to avoid a need for a separate surge chamber and to improve plant performance.
- Avoiding the abandoned mine workings by leaving a suitable buffer zone between the lower reservoir's outermost cavern and all known workings of the Mt. Hope mine.
- Selecting locations where excavation and ventilation shafts will avoid mine workings, will minimize surface visibility, and will accommodate existing and future quarry operations.
- Accommodating the in-situ stresses and geological structure of the host rock.

WATER CONDUIT SYSTEM

The water conduit system includes a series of shafts, tunnels, and draft tubes that pass water from the upper reservoir to the lower reservoir during generation and from the lower reservoir to upper reservoir during pumping.

The pressure shaft will drop 2800 feet (854 m) from the upper reservoir intake/outlet and join a horizontal tunnel which will manifold into six penstocks. The six draft tube extensions will be manifolded into conduits leading to the lower reservoir. The hydraulic performance of the manifolds and intake/outlet structures will be verified or adjusted based on the results of hydraulic model testing to be performed during final design.

Pump-turbine isolation spherical valves will permit each pump-turbine to be isolated from its penstock, which is connected to the common intake tunnel. These valves will prevent water leakage through the pump-turbine wicket gates during unit shutdown, provide emergency shutoff of flow through the pump-turbine if the wicket gates fail to close, and isolate the pump-turbine from the upper reservoir during maintenance.

The pump-turbines can be isolated from the lower reservoir by draft tube valves located at the draft tube exit or immediately below the pump-turbine runner. Current designs assume the use of spherical valves or butterfly valves for this purpose depending on the final location of the valves.

POWERHOUSE AND TRANSFORMER GALLERIES

The conceptual design provides for the six-unit powerhouse and a separate transformer gallery. Three bus duct tunnels plus the underground switchyard will interconnect the two major galleries. In the initial layout, the powerhouse is 74 ft (22.6 m) wide and 450 ft (137.2 m) long, with 60 ft (18.3 m) spacing between units. The transformer gallery is 49 ft (14.9 m) wide and 360 ft (109.7 m) long. These dimensions are expected to be refined during the final design process.

The powerhouse and transformer galleries will be connected to the main access shaft and the civil works shaft by additional tunnels that allow access and ventilation during construction and operation.

The dual-cavern arrangement was selected based on geotechnical considerations for ground support requirements and the construction schedule advantages of allowing multiple working faces. The dual-cavern scheme results in reduced individual width and length, as compared with a single cavern scheme. The smaller dimensions will significantly reduce the need for ground support for a given cavern height and allow easier handling of potential structural instabilities through conventional ground support methods. Because the individual cavern spans are less than those of a single cavern for this facility, the overall volumes of potential rock wedges are also significantly reduced, which in turn reduces ground support costs. The selected arrangement will be evaluated further as additional

geotechnical data become available.

The powerhouse cavern will accommodate the six pump-turbine/generator-motor units, two rail mounted bridge cranes, and necessary powerhouse service bays and assembly areas. The transformer gallery will house the main transformers, the transformer oil catchment sumps, high voltage and low voltage bus ducts, and possibly the silos providing access to the draft tube isolation valves.

The conceptual design calls for 9 three-winding, single-phase main transformers, plus one spare. Each bank of three transformers will serve a pair of generator-motors. This arrangement accommodates the main access shaft heavy lift hoist load limitations. One three-winding single-phase transformer is expected to weigh about 165 U.S. tons (182,000 kg) as shipped. This is comparable to the anticipated shipping weight of a generator-motor rotor. The alternatives would have been 6 three-phase transformers at 265 U.S. tons (292,000 kg) each or 18 double-winding, single-phase transformers. The first alternative would require too heavy a lift and the second would require a much larger transformer gallery excavation.

Each of the three bus duct tunnels will house the 21 kV bus duct and the major electrical equipment for a pair of generator-motors. Two of these tunnels will house the static frequency converter system to be used for rotating the units in air from standstill to full speed during pump starting.

The underground switchyard will receive the three 500 kV circuits from the three banks of main transformers. The two 500 kV circuits leading to the proposed utility substation 11 miles (18 km) away will originate from the switchyard. The use of an underground switchyard eliminates one 500 kV circuit to the surface. The switchyard will have a breaker and a half arrangement. The 500 kV circuits in the underground complex and the main access shaft will be enclosed in gas-insulated isolated phase bus ducts. At the surface, the system will be connected to overhead transmission lines.

PUMP-TURBINES AND GENERATOR-MOTORS

The major power generation equipment for this 2000 MW station consists of six reversible Francis type pump-turbine/generator-motor units. Each combined generator-motor unit will have the following general characteristics.

Nominal capacity	= 340 MW
Rotational speed	= 600 rpm
Number of guide bearings	= 3
Number of mode changes per day	= 20
Voltage	= 21 kV
Maximum Pumping Head	= 2657 ft (810 m)

The pump-turbines will be supplied by Kvaerner Energy and will be patterned after machines which have been used successfully at the Saurdal and Dinorwig Pumped Storage Stations. These will be the highest head single-stage pump-turbines in the world, as shown in Figure 3. The increase from the previous industry record is consistent with development trends in the recent past. The generator-motors will be enclosed in concrete barrels and will have water cooled stator windings.

SHAFTS

The conceptual design calls for five vertical shafts to connect the surface with the underground workings as follows:

1. Intake Shaft
2. Main Access Shaft
3. Civil Works Shaft
4. Lower Reservoir Excavation Shaft
5. Lower Reservoir Ventilation Shaft

The intake shaft is part of the water conduit system described above. The main access shaft, civil works shaft, and lower reservoir ventilation shaft will all remain open to the atmosphere following completion of construction. The lower reservoir excavation shaft will be decommissioned after construction is complete.

During construction, the main access shaft and the civil works shaft will both permit the inflow of air to be used in ventilating the construction areas of the powerhouse and transformer gallery complex and the lower reservoir. The main access shaft heavy lift hoist will be used for transporting all heavy permanent plant and construction equipment to the underground powerhouse and transformer gallery. Permanent access for personnel travelling to and from the powerhouse area will also be incorporated into the main access shaft. The civil works shaft will be used for personnel access to the construction areas and for removal of excavated rock from the powerhouse and transformer gallery complex, but not from the lower reservoir.

During plant operation, the main access shaft and the civil works

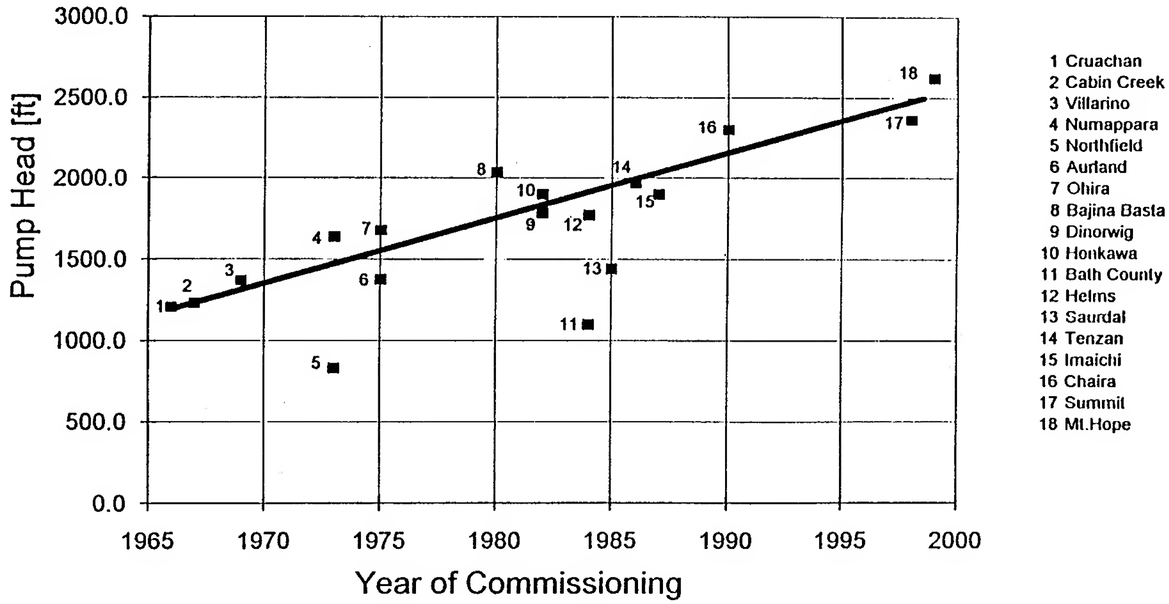

FIGURE 3 Industry Experience in the Development of Single-Stage Pump-Turbines towards Higher Heads

Source: Kvaerner

shaft will both serve only the powerhouse and transformer gallery. The main access shaft will house the heavy lift equipment hoist and the 500 kV buswork. This shaft will also provide ducted and free air intake for the powerhouse and transformer gallery complex. The civil works shaft will house an alternative personnel hoist and will provide for ducted and free air exhaust from the powerhouse and transformer gallery complex.

The construction of the lower reservoir will require that the lower reservoir excavation shaft be used to carry excavated rock to the surface at an average rate of 30,000 U.S. tons (33 million kg) per day. The magnitude of this effort requires this shaft to be positioned in a way which optimizes the ease of material handling at both the surface and underground levels. The shaft will be plugged and filled after construction to allow for future quarry activity.

The lower reservoir ventilation shaft will exhaust air during construction and act as the reservoir breather shaft during operation. This shaft has been positioned to minimize interference with the future quarry operations. The ventilation shaft will be provided with a hoist which will permit inspection and maintenance of the lower reservoir after construction is complete.

CONCLUSION

The Mt. Hope Waterpower Project will be innovative in many ways, but it will build on existing technology and proven experience. The major components -- surface excavations, underground powerhouse, major equipment shafts, and underground reservoir excavations -- have all been constructed in one form or another elsewhere. At Mt. Hope, these diverse experiences will be combined and advanced to provide a new dimension for pumped storage in the United States. Using Mt. Hope, the utilities which provide the regional electricity supply will have a new and powerful tool to help maintain the quality of electricity supply that a modern society demands.

Wanapum Development
Spillway Gate Hoist Failure

Raymond O. Ellis[1]
and
Richard V. Dulin[2]

ABSTRACT

The Wanapum spillway is controlled by twelve tainter gates, each 50 feet wide by 66.78 feet tall and weighing over 200 tons. On July 13, 1991 gate No. 7 fell from a raised position to the spillway crest due to a failure of the hoist system. Immediately afterward all spillway gates were inspected and it was found that gate No. 3 hoist system was undergoing failure. This paper will describe the District's strategy of hoist inspection, development of the hoist analysis, and its application, which resulted in the District saving millions of dollars by avoiding complete replacement of the hoists.

PROJECT AND EQUIPMENT DESCRIPTION

The Wanapum Development is located at River Mile 415 on the Columbia River in central Washington state, USA. The development includes a 1000 foot long, 10 unit, 996-MW hydroelectric powerplant; 5570 feet of embankment dam; and 823 feet of gated concrete spillway structure for a total dam length of 8537 feet. The dam develops a nominal gross head of 80 feet.

[1] Hydro Mechanical Engineer, Hydro Engineering Division, Public Utility District No. 2 of Grant County, P.O. Box 878, Ephrata, Washington 98823

[2] Senior Hydraulic Engineer, Stone & Webster Engineering Corporation, 7677 East Berry Ave., Englewood, Colorado 80111-2137.

The spillway gates are operated by a motor driven chain and sprocket drive hoist attached downstream of the gate face. The chain is double reeved, with the chain dead end attached to the hoist bridge structure. An electric gearmotor is located on one side, attached to a spur gear reducer, with a thru-shaft coupled to a 50-foot long cross-shaft, connected to the spur gear reducer located at the opposite machinery bridge. Refer to Figure 1 for an illustration of the hoist arrangement.

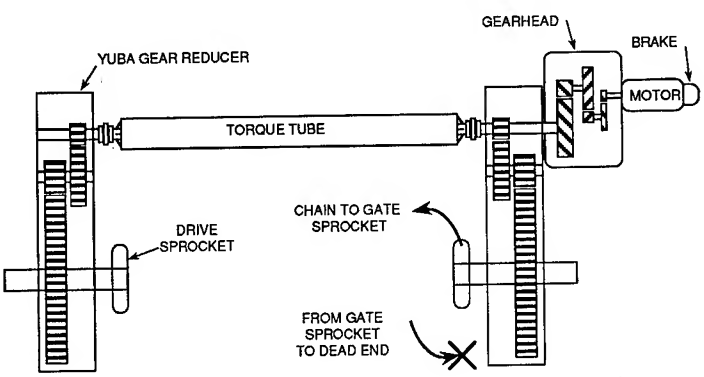

Figure 1. Spillway Gate Hoist Plan View

FAILURE SCENARIO

At 3:00 p.m. on Saturday, July 13, 1991, an operator was raising spillway gate No. 7 remotely from the control room. An erroneous position indication was received in the control room. Since the gate position indicators have malfunctioned in the past, another attempt to raise the gate was made. The gate indication changed rapidly and stopped at another erroneous position. Visual inspection of the gate showed that it was open approximately 2 feet. A further attempt to raise the gate resulted in severe vibration and noise being emitted from the hoist. The motor tripped out from thermal overload.

When the operator returned with an electrician the gate was found fully closed, the 50-foot long cross shaft had ejected from the hoist, the lifting chains were stripped off the hoist lifting sprockets and were found laying on the spillway apron. The gate was an estimated 4 feet open when the hoist system failed and dropped the gate in an uncontrolled manner.

GATE NO. 7 HOIST FAILURE INVESTIGATION

1. Structure

   The gate structure was inspected for cracked welds, distorted members and signs of distress. The inspection disclosed no notable distortion and the gate structure was intact. The gate had landed off center between the piers by approximately 1/2 - inch. Heavy contact loading on one of the side guide rollers had left its surface with a mushroomed appearance.

2. Hoist Components

   Examination of the hoist revealed extensive internal damage to the hoist gearmotor and cross shaft. The cross shaft was retrieved from the pool downstream of the spillway and was beyond repair. Many of the gears within the gearmotor had partial or completely broken teeth. The gear shafts were twisted, the bearing caps, retainers and support brackets were damaged. All of the teeth on the low speed pinion were damaged and completely stripped off. Examination of the low speed pinion indicated teeth failed from fatigue, some teeth exhibited spalling from preferential loading and misalignment. The inner cage bracket supporting the low speed pinion and low speed gear was assembled with excessively long bolts which bottomed out in their holes and permitted the cage to shift. The cage was not dowelled in place.

GATE INSPECTION AND HOIST ANALYSIS PROGRAM

The District investigated the extent of the hoist problems by proceeding with the following investigation:

1. Perform inspection of the remaining gate hoists, starting by draining the gearmotor's oil and inserting a borescope into the gearmotor housing to visually inspect the gears and other components.

2. Inspection of spillway gate No. 3 gearmotor uncovered parts of two broken teeth in the oil and teeth missing from the low speed pinion. Refer to Photo No. 1 for details of the low speed pinion. The gate was operated the previous day with no unusual noise. Disassembly of the gearmotor, parts revealed 12 out of the 16 teeth on the low speed pinion were damaged or completely removed. Serious spalling was noted on tooth surfaces. The teeth showed preferential loading, suggesting an alignment problem. Additionally, the spalling could be observed across three quarters of the tooth face suggesting a discrepancy in gear geometry.

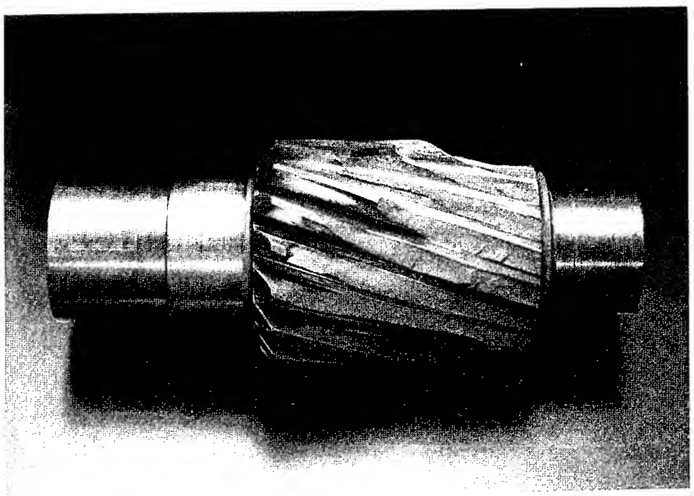

Photo 1. Gate #3 Low Speed Pinion

3. Retain the services of an independent engineering firm to perform a component-by-component review of hoist system design.

4. While the gearmotors were being inspected, the District was rebuilding the gearmotors in order to maintain the capability of the spillway. Concurrently with the gearmotor inspection and rebuild, the District had Stone & Webster Engineering Corporation analyze the hoist system.

DESIGN REVIEW

The spillway tainter gates are operated by a double reeved two part system. The drive side is identical to the idler side except for the primary reduction and motor. The gate attachment points are effectively at the outside edges of the gate. While the gate is raising, the dead load will remain equally distributed between the two sides. However the dynamic loads from side seal friction and binding of the gate may be excessive on one side and minimal on the other.

Hoist sizing is based on the required lift capacity and lift speed. The lifting requirements are linear in nature, however the hoist lifting capacity will increase in incremental steps. The lift speed is purely a function of the hoist's horsepower. The powertrain design is based on the torque requirements of lifting the load and the forces developed by the driving mechanism. If the powertrain was sized for the lift requirements and the

motor horsepower did not fall on the incremental horsepower rating, the next larger rated horsepower motor would normally have been installed. This will provide for an allowance of the required horsepower, however this excessive torque will need to be absorbed in the powertrain if the gate binds. Therefore, the hoist should be designed for the net capacity of the hoist, not the estimated hoist lift requirements.

At start-up or when the load binds, a squirrel cage motor can develop considerably more torque than at its rated value, depending on the motor design. The motor provided at Wanapum was a NEMA C design, which produced its maximum torque at locked rotor, 252% of the rated torque. This locked rotor torque is normally accounted for in the allowable design stresses in the equipment. This approach is straight forward in a single reeved system (one pick point). However in a double reeved system the lifting force can become unequally distributed when there is no means of equalizing the lifting force. These unequally distributed conditions were reduced into three load case scenarios: Ideal, Extreme and Severe and an analysis of the hoist was carried out for each case.

<u>Ideal Load Case</u>

In the Ideal Load Case the loads are equally developed between the two sides or pick points. In the event of a binding or frozen gate the locked rotor torque would be equally absorbed on both sides. The ideal load is not the calculated load requirements, but the lifting capacity of the hoist at the motor's rated capacity.

<u>Extreme Load Case</u>

In the Extreme Load Case it was assumed that an initial failure occurred on one side of the hoist and the gate is being raised by one side only. In this event all the motor's torque will be delivered to one side, and if the gate binds, locked rotor torque will be delivered. Effectively this means that each side of the hoist should be designed for 100% of the rated lift capacity. This approach would be failsafe, however it would require two times the rated hoist capacity to be applied to each side of the hoisting equipment. The component design factors provide a tolerance below the yield point of each component. If under this load case the stresses remain below the minimum yield point, component stresses may be acceptable.

<u>Severe Load Case</u>

In the Severe Load Case a design distribution of the rated lift capacity was developed. The rated ideal load will be distributed equally to both sides but the residual locked rotor torque will be absorbed on one side only. This assumption implies that the hoist rated capacity of each side should be

designed as a function of the maximum torque developed by the primary reduction, less the rated capacity of one side, divided by the ratio of the maximum potential torque and the rated torque. In the case of the existing hoists at Wanapum, each side of the hoist should have been designed based on the severe load case at 160% of the rated lift capacity when using a helical gearmotor and 150% of the rated lift capacity when using a wormgear reducer as the primary reduction. The 10% reduction in the rated capacity when using the wormgear is due to the worm's lower efficiency at maximum potential torque.

MOTOR AND PRIMARY REDUCTION

The motor and the primary reduction in the Wanapum hoist, as in most hoists, should be designed for locked rotor or maximum torque conditions. The American Gear Manufacturers Association (AGMA) for gearmotors has developed an "Application Classification" system as a function of the "Maximum Peak" developed by the driving mechanism. This method will reduce the allowable stresses as a function of the maximum torque delivered by the driving mechanism. There are three Classes which represent a reduction in the allowable stresses as a direct function of the maximum peak: Class 1, 200%; Class II, 280%; and Class III, 400% of rated torque. The gearmotor provided at Wanapum had a Class I rating, however the motor provided was a NEMA C design which produced a maximum torque of 252% of the rated torque. The gearmotor should have fallen under the Class II designation. The failure at Wanapum cannot be solely attributed to the 26% increase in allowable stresses, since loss of alignment of the low speed pinion bearing cage was the most probable cause. However, the designer should address the maximum torque requirements throughout the hoist's design.

The design of a gate hoist must address the potential of a gate binding, from alignment problems to simply freezing in place during winter. The helical gears provided in the existing gearmotor are efficient in transferring torque at rated speed and locked rotor. This is an advantage in most applications, however this must be accounted for in the entire system. A wormgear as the primary reduction does result in a lower overall efficiency in the hoist system, i.e. an increase in the hoist required horsepower, however this will reduce the overall design requirements of the gate hoist. When the gate first starts to move the wormgear efficiency is low, approximately 40% for a 60:1 reduction. This efficiency will increase nearly two fold as the motor speed increases to rated speed. This provides for a flattened torque curve since the hoist motor's torque is normally high at start-up and decreases as the motor approaches rated speed. The nature of the wormgear efficiency effectively provides for a soft coupling between the primary reduction and the balance of the system. Refer to

Figure 2 for a graph of torque versus efficiency of the prototype primary reduction.

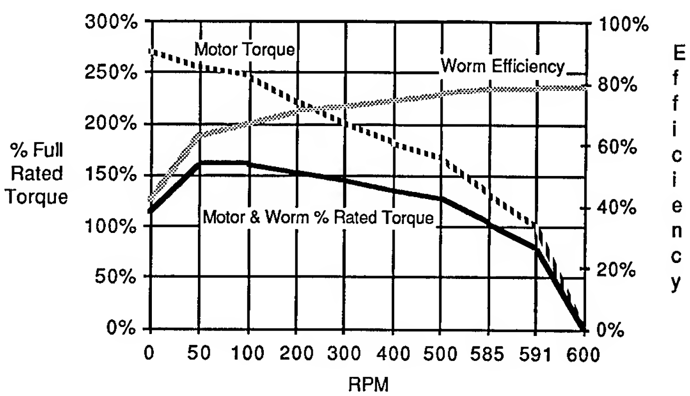

Figure 2. Worm Output as a Function of RPM

The wormgear does provide a secondary benefit. At minimal reduction ratios the wormgear is self-locking when static and at higher reduction ratios will provide a braking action when the motor is stopped. At Wanapum the helical gearmotor was not self-locking so the entire gate weight and the momentum of the gate and hoist must be absorbed by the motor brake.

SECONDARY REDUCTION

The secondary reduction was reviewed based on the load cases presented above. Under the ideal case the stresses were below the allowable stress level. The extreme load case did provide for stress levels above the yield point, however this case is considered unrealistic for a design basis. The severe case produced acceptable stress levels when using a wormgear in the primary reduction. When utilizing the existing helical gearmotor as the primary reduction several of the components were near the yield point.

PLANS FOR RESTORATION

In the interim, the District had seven of the gearmotors rebuilt as a precaution until the analysis could be completed and installation of the prototype hoist system was underway. A detailed inspection was

performed with any anomalies noted. Any problems discovered that might prevent reliable operation were corrected. All bearings were replaced, the bearing cage brackets suspected of causing the low speed pinion misalignment were doweled. The motor brakes were inspected and adjusted to manufacturer's tolerances.

Since the gates had provided almost 30 years of reliable service, the margin of safety could be realized by replacing the gearmotors with another type of primary reduction system at a nominal cost. The District concluded that this approach with a wormgear provided an acceptable degree of risk at a substantial cost savings. The hoist should be designed to lock-up under a severe load case and trip out the breakers without the failure of the hoist system and dropping of the gate in an uncontrolled manner.

The District chose to install a prototype hoist arrangement at gate No. 7 prior to installing a new primary reduction system on all 12 gates. The prototype permitted the District to test and fully evaluate the new system prior to investing significant funds. The actual zero-speed efficiency critical to the design of the new wormgear system was confirmed to be 27% less than originally estimated. However, the motor torque curve for the 600 rpm motor at locked rotor was 19% greater than anticipated. This combined for a net reduction in torque of 38% at locked rotor with respect to the gearmotor.

In addition to the replacement of the primary drive system, the District intends to replace the gate rollers and install an electric heater system in the gate side seal bulbs to reduce the potential of the gate freezing to the structure.

Two out of eight gate side rollers on gate No. 7 were binding and would not rotate without application of extreme force. These rollers had graphite impregnated bronze bushings of the type employed during the design of the gates. The graphite lubricant washed out of the buttons and combined with debris would swell and bind. The District replaced these rollers with a design combining 100% teflon lubricated bronze bushings on a more robust roller and bracket assembly.

During periods of extreme cold, the gate structure, lifting chains and seals accumulate a substantial quantity of ice. On a few occasions the gates were frozen to the spillway structure and the ice had to be broken off with a crane and wrecking ball. The District will utilize a method obtained from the Corps of Engineers, the installation of heater strips in the gate seal bulbs to reduce adhesion of the seals to the piers. This system is reported to work well even at extreme sub-zero periods where embedded heater strips would not have sufficient capacity to be effective.

PROTOTYPE PERFORMANCE AND CONCLUSION

Installation of the prototype hoist system took place in the Fall of 1992 without incident and no major construction difficulties were experienced. Two additional changes were implemented during the course of the work: replacing gate idler sprocket bearings and reversing the gear faces in the Yuba gear boxes. The total cost of Contract work for the prototype was approximately $100,000. Replacing bearings in the Yuba gearboxes, combined with installation of new guide roller bushings and replacement of the gate lifting sprocket and idler sprocket bearings had a profound effect on the ease and smooth operation of the hoisting equipment.

Prior to this work on the prototype, all of the gates emitted a loud "bang" noise at random intervals when raising and lowering the gates. This noise could be heard for several miles and has apparently been present since the gates were originally installed. Some suspected the guide rollers were the cause. Others suspected the idler sprocket bearings, the lifting chains or the gate trunnion bearings caused this noise. After modification, the gate was lifted to the full raise position twice, (with a floating bulkhead in place) to confirm gate operation with the prototype hoist and no abnormal noise was observed. The District is still unable to determine the source of this noise.

By replacing the spillway gate hoist helical gear primary reducers with a wormgear type system, the District will realize several significant benefits. The wormgear system has improved braking capability, and the gates will operate smoother and quieter. The most significant advantage is the reduction of stresses if the gate hoist when overloaded with ice or binding of the gate creates unequal loading on the lifting chains. Applying the reduction in stresses on the hoisting system because of the lower efficiency of the wormgear system will result in a substantial cost savings for the gate rehabilitation project. The alternative would be complete replacement of the hoists which would increase the cost of the gate rehabilitation by over $10 million.

Structural and Hydrologic Considerations for the Flooding Reservoir Operations of Jiguey Dam, Dominican Republic

Guy S. Lund, M.ASCE[1] and Ed A. Toms[2]

Abstract: This paper discusses the structural and hydrologic evaluation performed prior to the initial filling of Jiguey Reservoir. Jiguey Dam is located on the Rio Nizao about 100 kilometer west-northwest of Santo Domingo, Dominican Republic.

A three-dimensional finite element analysis was performed to determine the expected stresses and deflection of the dam during initial filling of the reservoir. This paper discusses the assumptions used to define the numerical model and simulate the expected loads during filling. The results from the analysis were used to produce deflection envelopes, which were used during the filling of the reservoir as a basis for interpretation of instrumentation measurements.

A hydrologic analysis was performed to determine the maximum allowable rate of rise for the reservoir. The paper discusses the parameters and methodology used to perform the study. The analysis shows that the dam has inadequate outlet capacity to regulate the anticipated storm events, which may result in a rate of rise greater than allowable.

INTRODUCTION

The Rio Nizao system consist of three tandem multiple purpose reservoirs: Jiguey; Aguacate; and Valdesia. Jiguey and Aguacate dams were added in 1991 to increase the hydropower capacity and stored water of the system for domestic and agricultural use.

Jiguey Dam is a two-centered concrete gravity arch dam with gravity thrust blocks on both the right and left abutments. The dam is 338.4m long with a maximum structural height of 115m. The dam consists of an 85m long left gravity thrust block section, a 193.4m long two-centered gravity arch section, and a 60m long right gravity thrust block section. The two-centered gravity arch section consists of a 122.5m long left overflow spillway section with a 150m radius and a 70.9m long right non-overflow section with a 120m radius. The typical gravity arch section has a crest thickness of 6m and a vertical upstream face. The

[1]Civil/Structural Engineer, ECI, 5660 Greenwood Plaza Blvd., Suite 500, Englewood, Colorado 80111, (303) 773-3788

[2]Hydrologic/Hydraulic Engineer, ECI.

downstream face is vertical from the crest El. 554.5m to El. 543.5m and then slopes at 0.6 Horizontal (H) to 1.0 Vertical (V).

The typical thrust block section has a crest thickness of 6m at El. 554.5m. The upstream face is vertical. The downstream face is vertical from the crest to El. 546.0m and then slopes at 0.8 H to 1.0 V.

A 6,900m long power tunnel transfers the water from Jiguey Reservoir to a 98 MW underground powerhouse. Aguacate Dam, a power regulating dam downstream of Jiguey Dam, is a 51.5m high concrete gravity dam. A 11,000m long power tunnel transfers the water from Aguacate Reservoir to a 52 MW underground powerhouse. Valdesia Dam located downstream of Aguacate Dam is a 82.0m high concrete gravity dam with domestic and agricultural diversions and a 54 MW powerhouse.

STRUCTURAL CONSIDERATIONS

Stress Analysis A general linear elastic finite element analysis was used to determine the stresses and deflections of the dam. The dam was modeled using 216, variable node, isoparametric thick shell elements with two elements through the thickness. A significant portion of the foundation was modeled using 630 eight node, isoparametric thick shell elements with eight elements through the thickness from upstream to downstream edge. The foundation extended at least two dam heights upstream of the dam, and one dam height downstream and into the abutments. The finite element model is shown in Figure 1.

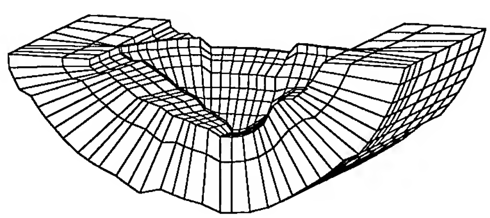

Figure 1 - Finite Element Model of Jiguey Dam.

Material Properties. The material properties used for the concrete in the model were based on field data collected during construction and the engineering guidelines specified by the U.S. Bureau of Reclamation (USBR). Concrete cylinders, collected during placement of the mass concrete in the dam, were tested to determine the 7- and 28-day unconfined compressive strength of the concrete. These strengths were compared to the strengths for a typical rate of strength gain curve for type-I cements, as reported in the USBR Concrete Manual, and used to estimate an average 1-year unconfined compressive strength. The instantaneous concrete modulus of elasticity was computed in accordance with guidelines of the American Concrete Institute (ACI), based on the estimated 1-year unconfined compressive strength. The sustained modulus of elasticity was assumed equal to 70 percent of the instantaneous modulus, based on USBR criteria. The material properties assumed for Jiguey Dam are summarized in Table 1.

TABLE 1
Material Properties

Properties	Values	
Compressive Strength	3,500	tons/m^2
Tensile Strength	210	tons/m^2
Modulus of Elasticity		
Sustained	2,100,000	tons/m^2
Dynamic	3,000,000	tons/m^2
Poisson's Ratio	0.20	
Density	2.403	tons/m^3
Thermal Expansion	0.000,009	m/m/°F
Reservoir Unit Weight	1.0	tons/m^3

The average Rock Quality Designation (RQD) values of foundation core samples from the Jiguey damsite ranged from 10 percent to 60 percent. The higher RQD values were located near the base of the dam. These RQD values suggest that the foundation deformation modulus would vary from less than 70,300 tons/m^2 at the dam crest to 422,000 tons/m^2 at the base.

Previous geoseismic investigations indicated that the deformation modulus would vary from 1,500,000 tons/m^2 at the base of the dam to 100,000 tons/m^2 at the dam crest.

The foundation deformation modulus values varying from 1,054,800 tons/m^2 at the base of the dam to 210,000 tons/m^2 at the dam crest were selected based on review of the geologic data. The precision of foundation material property data was low, therefore, a range of values for foundation deformation modulus was analyzed. The minimum foundation deformation modulus values was assumed to be 50 percent of the reference deformation modulus values. The maximum foundation deformation modulus values was assumed to be 200 percent of the reference deformation modulus values. The assumed foundation properties are summarized in Table 2.

TABLE 2
Foundation Properties

Properties	Values
Deformation Modulus	
Minimum	50 percent of Reference
Maximum	200 percent of Reference
Poisson's Ratio	0.20
Internal Angle of Friction	45.0 Degrees
Cohesion	35.1 tons/m^2

The foundation for Jiguey Dam was over-excavated to remove unsatisfactory material, resulting in the use of dental concrete to shape the abutment underneath the left and right gravity thrust block sections. The finite element model used the average dam/foundation contact between the upstream heel and the downstream toe of the dam to define the dam profile. Portions of the left gravity thrust block section have a significant volume of dental concrete below this assumed dam/foundation contact. The dental concrete below the dam/foundation contact was assumed to behave similar to the foundation, therefore, the material properties were assumed equivalent to those for the foundation.

Loads. The finite element analysis can be used to investigate numerous loads acting on the dam. In this study, each load was applied independently to the model to study its effect on the structure. This provides a method for verifying the combined loads and for determining the loads that contribute the majority of stress and deflection to each load combination. Each load combination was analyzed to study the effect of the minimum and maximum foundation deformation modulus . The loads studied in the analysis are summarized in Table 3.

TABLE 3
Static Loads

Load	Description
Gravity	Dead weight of dam.
Temperature	Loads caused by volumetric changes in the concrete due to concrete heat of hydration and variations in the reservoir and air temperatures.
Reservoir	Hydrostatic water pressure applied to the upstream face of dam.
Uplift	Internal hydrostatic water pressure applied along the dam/foundation contact.

The gravity loads were analyzed using the staged construction method. This method simulates the construction of the dam, analyzing the concrete placed during the year and it effect on the foundation and previously place concrete. The analysis was performed in three phases, one phase for each year of construction. The concrete was model to the elevation corresponding to concrete placed during the year.

Preliminary analysis of the temperature loads evaluated temperature corresponding to 3-, 4-, and 5- years after placement of the concrete. The results determined that the most severe load would occur for the temperature corresponding to 3-years after concrete placement. The initial filling of the reservoir more closely corresponded to the 3-year temperature loads. Therefore, the temperature loads corresponding to 3-years after concrete placement were used in the analysis.

Four reservoir elevations were analyzed to gain an understanding of the dam behavior during the filling of the reservoir. The reservoir water levels included in the analysis are minimum normal operating lever El. 500m, an intermediate reservoir level El. 525m, maximum normal reservoir lever El. 541.5m, and the PMF reservoir level El.555.5m.

The uplift load was applied along the dam/foundation contact and followed the FERC guidelines*(FERC, 1991)*. For the uncracked base analysis the uplift load was assumed equivalent to a linearly varying pressure from full reservoir head at the upstream heel of the dam to a reduced head at the drain location, and a linearly varying pressure from the reduced head at the drains to the tailwater head at the downstream toe of the dam. The reduced head at the drains was assumed equal to tailwater head plus one-third of the difference between full reservoir and tailwater head.

Stress Analysis Results Based on the results of the analysis the dam does not entirely behave as a three-dimensional structure. In the upper portion of the dam the horizontal arch stresses indicate that the dam's vertical contraction joints will open due to the effects of temperature loads. In the lower portion of the dam the horizontal stresses show that the vertical contraction joints are closed, thus preserving some three-dimensional behavior at lower El.s of the dam.

All of the compressive and most of the tensile stresses predicted by the finite element model are less than the allowable limits. Some tensile stresses predicted by the finite element model on the upstream face dam near the dam/foundation contact are due to the geometrical irregularities of the foundation. The dam/foundation contact is unable to develop tensile stress, therefore, the foundation will relax and eliminate, or greatly reduce, the tensile stress in the concrete.

The behavior of the dam is influenced by the assumed values for foundation deformation modulus. An increase in the modulus increases the stiffness at the dam/foundation contact, which results in an increase in the cantilever action of the dam and a decrease in the arch action of the dam, as shown in comparing Figures 2 and 3.

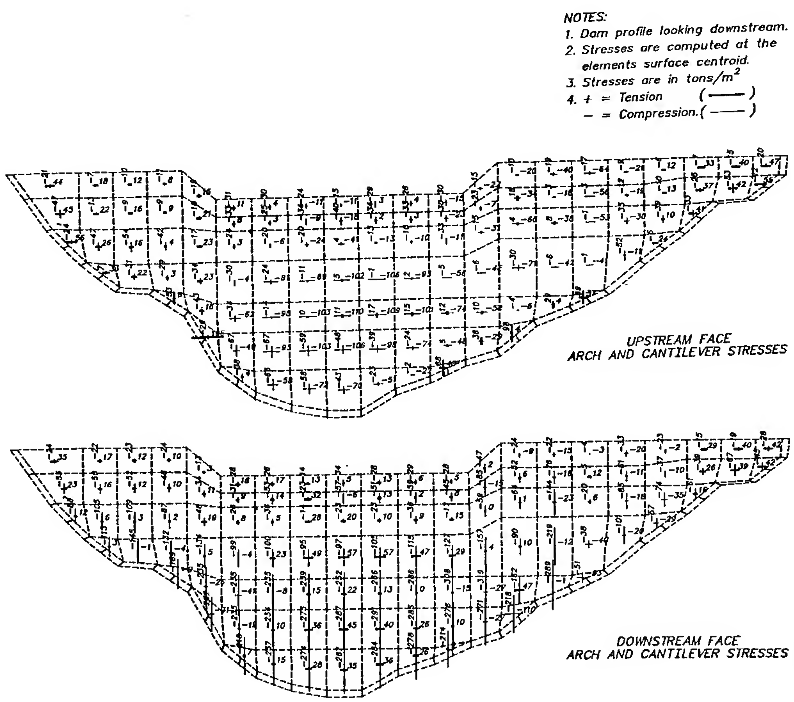

Figure 2 - PMF Reservoir El. 555.5, Minimum Foundation Modulus

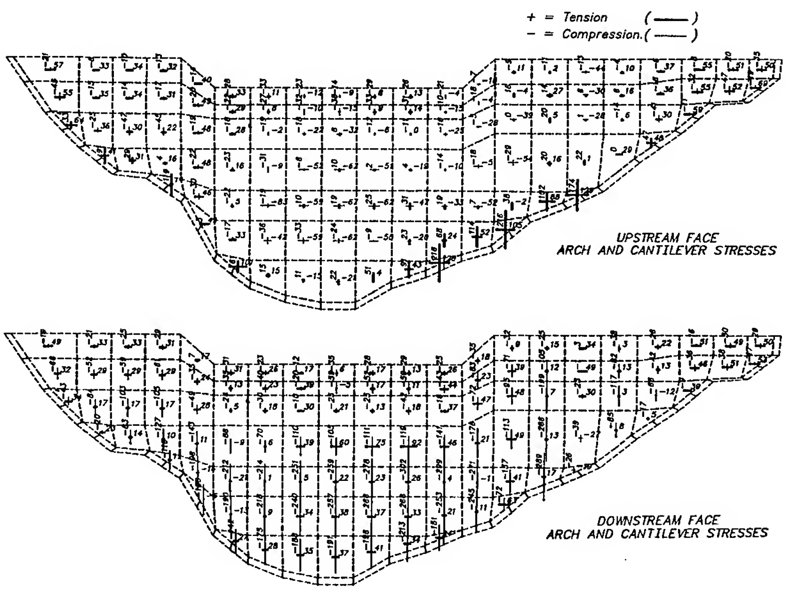

Figure 3 - PMF Reservoir El. 555.5, Maximum Foundation Modulus

Deflection Results The deflections output by the finite element analysis were resolved into a local element coordinate system corresponding to the elements vertical and radial directions. The radial direction corresponds to the deflection perpendicular to the axis of the dam. The deflections were used to develop deflection envelopes, plots comparing the deflection due to the maximum and minimum foundation modulus values. The deflection envelope along dam gallery at El. 525m for the PMF reservoir El. 555.5m is shown in figure 4.

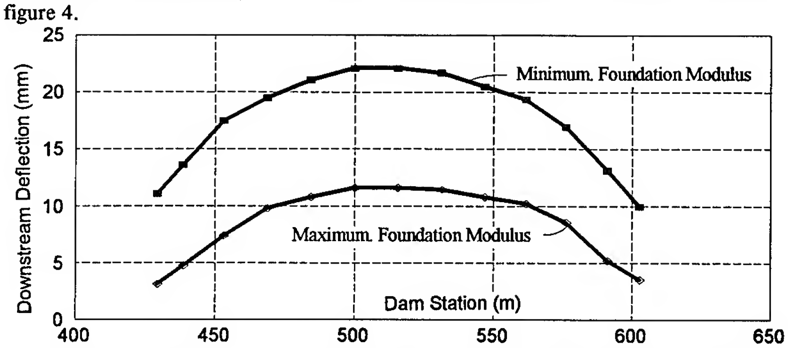

Figure 4 - Deflection Envelop along the Dam Crest for PMF Reservoir El. 555.5

HYDROLOGIC CONSIDERATIONS

Introduction. Hourly operations for Jiguey reservoir, due to certain flood events, were simulated to develop operating rules to control the rate of filling from reservoir elevations 500m to 520m and 520m to 541.5m. Valdesia and Aguacate Reservoirs, which both have gated spillways with adequate capacities to pass maximum flood routed, were operated to maintain maximum operation levels.

Hydrology. The 2-, 5-, 10-, 25- and 30-year storm events were used to perform hourly (flood) reservoir operations. It was considered unlikely that a storm greater than the 30-year event, with a 3.33% chance of developing, would occur during the 12 month initial filling period.

The shape of each storm hydrographs was based on the 1989 Hurricane Gilbert, which produced a peak discharge of approximately 950 m^3/sec for the Jiguey watershed. The hurricane was estimated to be a 25-year storm event by comparing the actual measured peak of the storm to storm peaks with frequencies previously developed for the river. Different frequency storm hydrographs for Jiguey watershed were developed by scaling the storm hydrograph ordinates of Hurricane Gilbert to obtain a peak discharge equal to that determined by the storm frequency analysis for a given frequency of occurrence.

Local inflow hydrographs, generated by the contributing area between Jiguey and Aguacate Dams and Aguacate and Valdesia Dams, were estimated by the net area ratios of each watersheds. These ratios were used to scale the ordinates of the storm hydrographs generated for Jiguey Reservoir.

The procedure used to compute the inflow hydrographs was developed during the initial planning stages of Jiguey Dam. It was deemed a conservative approach in estimating the hydrologic response of the contributing watersheds for the Jiguey-Aguacate-Valdesia Reservoir System.

Jiguey Reservoir. The total volume of Jiguey Reservoir is 168 MCM at normal maximum operating El. 541.5m. The volume is divided into storage components separated by control elevations as shown on Figure 5. The dead storage is from the river bed El. 449.8m to the low level outlet El. 479m. Reservoir releases are not possible below this elevation. Inactive storage is from El. 479m to the minimum operating El. 500m. Hydropower production is not possible below this elevation. Active conservation storage is from the minimum operating El. 500m to the spillway crest El. 541.5m. The exclusive flood control storage is not available for Jiguey Reservoir.

Reservoir Releases. Releases from the active conservation storage normally will be made through the low level outlets during the initial filling period. The power tunnels can be utilized when the hydropower facilities are completed.

The low level outlets also will be utilized during flooding operations between El. 479m and the spillway crest El. 541.5m. Flows would discharge freely over the uncontrolled spillway crest above El. 541.5m. The low level outlets will not be operated when the spillway is being utilized to avoid wave formation in the spillway plunge pool.

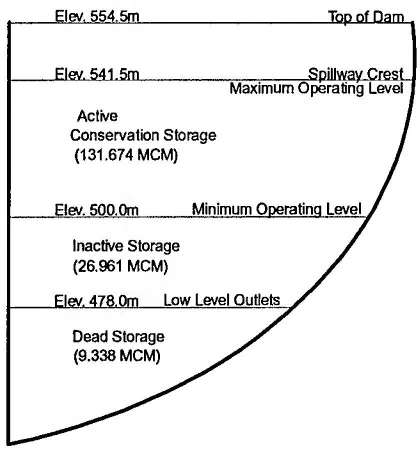

Figure 5 - Jiguey Reservoir Storage

Maximum Rate of Rise. The maximum allowable rate of rise was established based on selected reservoir elevations 500m, 525m, and 541.5m, which were analyzed in the structural stress analysis. The rate of rise provides time to monitor and evaluate the reservoir system. It is important to ensure that the dam, abutments, and other operating systems are responding as expected to the imposed reservoir loads. The rate of filling will be uncontrolled from the river bed to El. 500m. The rate of filling will be 3m per day from El. 500m to 520m and 1m per day from El. 520m to 541.5m. The reservoir level will be held at El.s 500m and 525m for one month for more extensive monitoring and evaluation of the system. Figure 6 illustrates the maximum filling rate criteria.

The filling of Jiguey Reservoir is regulated by the low level outlet works, with the gate sill located at El. 478m. The capacity of the low level outlets is 134 m^3/sec at El. 541.5m, which limits regulating control during high volume inflows. High runoff rates in the watershed areas, which exceed the available discharge capacity of the low level outlet works, may result in an accelerated rate of rise in the reservoir in excess of the maximum allowable, or the inability to maintain the defined holding El.s. In the event of high inflow, the low level outlet works will be operated to minimize the rise, and to lower the reservoir to the predefined holding elevation following a flood.

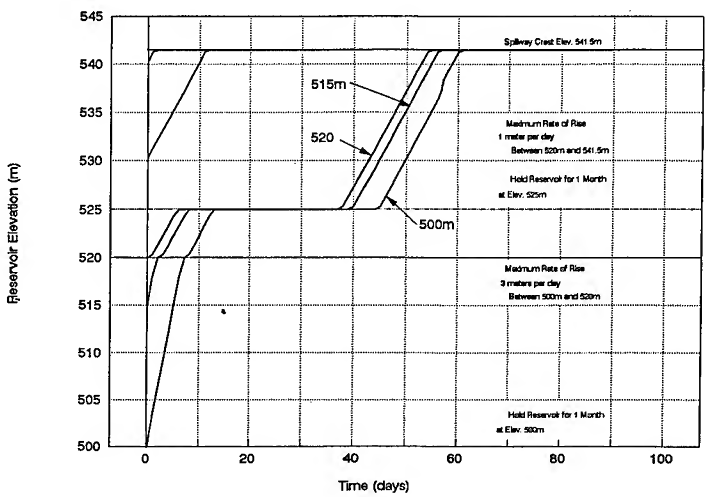

Figure 6 -Allowable Maximum Rate of Rise for Jiguey Reservoir.

Simulated Hourly Reservoir Operations. Five reservoir elevations were selected for the simulated hourly operations; 490m, 511m, 523m, 533m, and 541.5m. The minimum El. 490m is approximately half the distance between the low level outlets and the demarcation line between the unlimited and 3m per day maximum rate of rise. There is about 146.6 MCM between reservoir El. 490m and the spillway crest El. 541.5m. Elevations 511m, 523m, and 533m divide the 146 MCM volume into approximately four volumes of 36.6 MCM.

Aguacate and Valdesia spillway gates have the capacity to pass the studied storm events at their respective maximum operating elevations. For this study, Aguacate Reservoir was assumed to be at maximum operating El. 328m and Valdesia Reservoir was assumed to be at minimum operating El. 130.75m.

The filling rule curve for the different starting reservoir elevations was developed from the hourly operations and is presented in Figure 7.

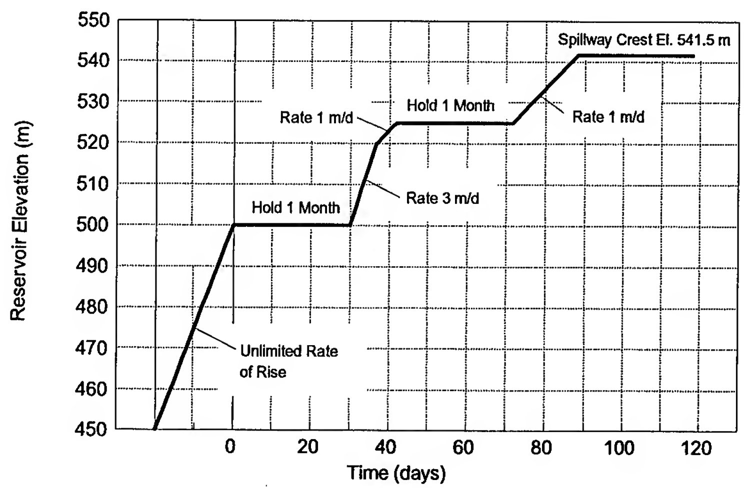

Figure 7 - Filling Rule Curve for Jiguey Reservoir.

Conclusions

The flooding operations during the initial filling period were developed using the Hurricane Gilbert storm event. The filling rule curves were developed based on three-dimensional stress analyses of the dam and abutments. The discharge capacity of the low level outlets is small and limits the ability to regulate the reservoir. Excessive flow caused by storm events at Jiguey Dam could result in accelerated filling greater than the maximum filling rule curve. An accelerated filling rate is not expected to have a detrimental effect on the dam integrity based on the results from the three-dimensional stress analyses. Every effort should be taken to anticipate a storm event and to lower the reservoir to the maximum filling rule curve El. after the storm event.

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A Seven-year Review of Bath County Power Tunnel Performance

K. L. Wong[1], A. M. Wood[2], D. E. Kleiner[3]

ABSTRACT:

Three high head (1260 ft) power tunnels are part of the Bath County Pumped Storage Station, jointly owned by Virginia Power and Allegheny Generating Company. The station is located 70 miles north of Roanoke, Virginia. It is a six unit 2100 MW installation and is presently the largest in the world. The station has been in continuous service since December 1985. During the initial filling in the spring 1985, excessive tunnel leakage and penstock steel liner buckling necessitated a remedial program. The remedial work consisted of high pressure grouting (600 psi), adding 1400 drains, and adding instrumentation to monitor pressures, flows and displacement. During the past seven years of successful operation, the station has maintained a team providing daily surveillance and regular drain maintenance. This paper presents a review of the tunnel and penstock performance and an update of observations and instrumentation data evaluation reported in 1988 (Reference 3).

1 PROJECT DESCRIPTION

The power station consists of the upper and lower reservoirs, power tunnels and a powerhouse. The reservoirs are contained by embankment dams (Figure 1). Water is conveyed through three 28.5-foot (8.7 m) diameter, concrete-lined power tunnels. Each has an upper and lower horizontal segment approximately 3100 to 3600 feet (966-1122 m) long, connected midway by a 990-foot (302 m) vertical shaft. Each lower tunnel bifurcates into 18-foot (5.5 m) diameter, reinforced concrete and steel-lined penstocks that are 900 to 1260 feet (280-392 m) long. Each penstock

1 Associate and Section Head, Harza Engineering Company, Chicago, IL.
2 Supervisor, Geotechnical and Environmental, Virginia Power, Warm Springs, VA.
3 Vice President, Harza Engineering Company, 233 South Wacker Drive, Chicago, IL.

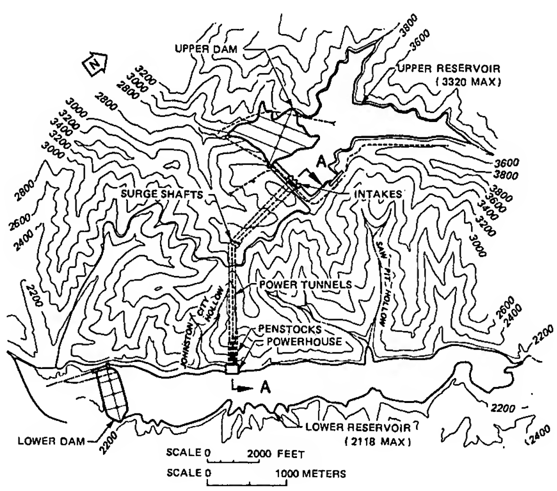

FIGURE 1. BATH COUNTY PUMPED-STORAGE PROJECT PROJECT PLAN

supplies a pump/turbine in the powerhouse located on the west bank of the lower reservoir.

2 PROJECT GEOLOGY

The Station is located at the northwestern margin of the Appalachian Fold Belt of western Virginia. Devonian age rock units in the project area are predominately thinly bedded siltstone and shale of the Millboro/Brallier formation in the lower reservoir, and blocky sandstone and siltstone of the overlying Chemung formation in the upper reservoir (Figure 2).The interface of the two formations is at the mid-height of the vertical shafts. The structural trend is northeast-southwest, paralleling the trend of the Appalachian Mountains. Rock units are buckled in tight chevron folds in the vicinity of the powerhouse and penstocks, but strata in the remainder of the tunnel route generally dip at very low angles (5-10 degrees) to the northwest.

3 FILLING

The initial filling of the upper reservoir using only Tunnel 1 started in the spring of 1985. The water level reached the upper reservoir and created an internal tunnel pressure of about 1000 feet (305 m). Tunnel 1 was estimated to be leaking 7700 gpm (480 l/s). A 44-foot (13.5 m) section of steel liner in Penstock 3 buckled and ruptured. Tunnel 1 was unwatered for

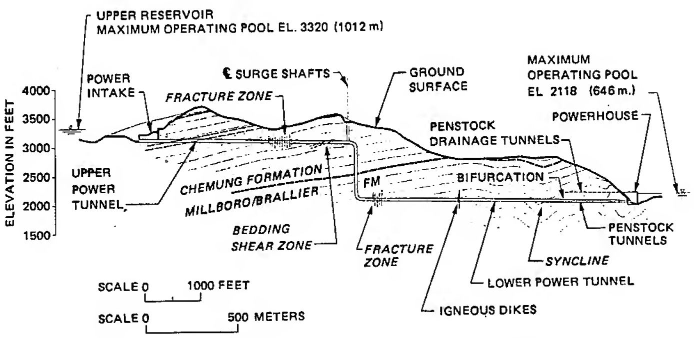

FIGURE 2. BATH COUNTY PUMPED-STORAGE PROJECT
SECTION A-A GEOLOGY ALONG POWER TUNNEL No. 2

remedial treatment which consisted of grouting, adding drains and instrumentation.

The second filling with all three tunnels started on September 12, 1985. The upper reservoir reached its maximum operating level (El.3320) on December 10, 1985. The six generating units were commissioned later that month.

4 GROUTING, DRAINAGE, AND INSTRUMENTATION

A multi-phase program of grouting, drainage, and instrumentation was developed to reduce, control, and monitor seepage from the tunnels. The program was designed to reduce rock mass permeability, increase the modulus of deformation adjacent to the tunnels, and monitor seepage and pressures. The treatment was completed in the summer 1985. Additional drainage was installed in 1986 and 1987 (References 1 and 2).

4.1 Grouting

The grouting work included a complete ring contact and consolidation grouting program for the lower horizontal run and lower portion of the vertical shaft of each tunnel. Grouting pressures reached 600 psi (4140 kPa).

4.2 Drainage

Drain holes and relief wells were installed from 1985 through 1987. Sixty-three 6.5-inch (165 mm) diameter relief wells were drilled to the south of the tunnel corridor. Thirty-one wells were installed to the north.

A drainage system was installed for the penstocks which included parallel drainage tunnels 140 feet above the penstocks, a zig-zag tunnel over the upstream ends of the steel lined penstocks, and a tunnel over the access tunnel (Figure 3). Drain holes were drilled from these tunnels to form drain curtains.

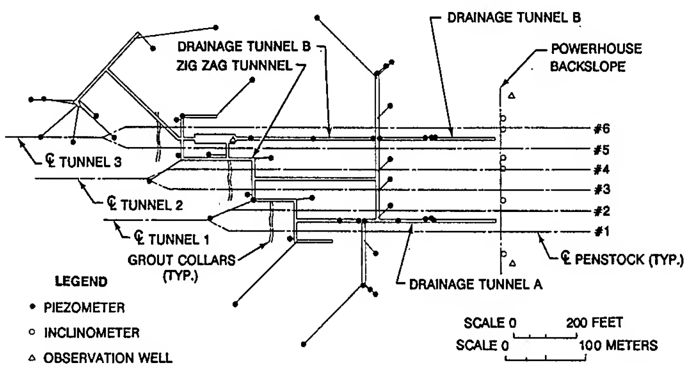

FIGURE 3. BATH COUNTY PUMPED-STORAGE PROJECT PENSTOCK DRAINAGE TUNNELS - PLAN

4.3 Monitoring

Forty-nine piezometers (Figure 3) were installed from the penstock drainage tunnels to monitor hydrostatic pressures in the rock adjacent to the tunnels and penstocks. Sixty-one piezometers were also installed along the tunnel route and on either side of the tunnels to measure pressures within the mountain. The instrument readings are collected using an automatic data collection system.

Three inclinometers, three multiple point extensometers, and 24 span gauges were installed in and near the penstock drainage tunnels. Eighteen

permanent, V-notch weirs were placed to monitor surface flow throughout the project area from tunnel seepage.

Station personnel routinely inspect surface conditions, drainage tunnels, embankment slopes, and cut slopes for evidence of hillside instability, new springs, or other seepage related phenomena. The station also uses periodic survey measurements, some to first order precision, to detect displacement.

4.4 Drain hole maintenance

Calcification of drain holes poses a threat to the long term effectiveness of the drain holes. Since June 1987, the station has been maintaining 880 of the 1320 drain holes in the penstock drain tunnels. The intent is to remove locally accumulated silt in rock joints and the early stages of calcite formation before hardening become pervasive.

A video camera is used to assess conditions and sources of flow in drain holes. All drains included in the program are pressure cleaned using a 5000psi water blaster (hydrobrooming). Selected drains are then back flushed by setting a pneumatic packer at specific intervals within the drain and injecting water at pressures up to 450psi (hydroflushing). Drains that have mineral build up, that cannot be removed by the water blaster, are reamed. Down hole pressure data obtained during back flushing operations is used to supplement piezometer data and evaluate long term seepage and pressure trends in the vicinity of the penstocks. To date more than 3250 hydrobrooming operations, and 5700 down hole pressure measurements have been taken as part of the hydroflushing procedure. Drain hole flow increases and porewater pressure decreases are occasionally observed after flushing.

The total flow from the penstock drainage tunnels has remained relatively stable since 1987, indicating maintenance activities are effective at keeping the drainage system in optimum condition. It should be noted that the distribution of flow has changed within the system with a larger number of the lower flowing drains (<5gpm) typically decreasing in flow, while a small number of higher flowing drains have increased in flow. The increases in flow of the higher flowing drains has masked the decreases noted in the lower flowing drains.

The maintenance crew that performs the drain hole maintenance activities includes personnel fully capable of drilling new drain holes or grouting shut existing drains.

5 TUNNEL PERFORMANCE

5.1 Piezometric Heads in the Mountain Mass

The minimum horizontal in-situ rock stress in the lower tunnel area is estimated to be about 84% of the tunnel operating pressure of 575 psi. Table 1 presents the results of 7 piezometers along the tunnels at about the tunnel level (El. 2000±). The piezometric head recorded at each piezometer is presented as a percentage of the maximum operating tunnel head. The distance between the tunnels and piezometer is also indicated.

It is evident that the recorded piezometric pressures in the mountain mass outside the tunnel area are less than the in-situ rock pressures. Hydro jacking outside the grouted zone, if occurring, appears to be safely confined deep in the mountain. In addition, the results have indicated the piezometric pressures in rock have gradually decreased over the years.

Surface weir data collected regularly over the years also indicates decreasing trends. The total tunnel leakage has decreased from 6000gpm (378 l/s) in April 1987 to 5200 gpm (328 l/s) in January 1992.

Table 1
Piezometric Heads Along Lower Power Tunnels

Piezometer	Tunnel #2 Station	Distance from Tunnels (feet)	Piez. Head/ Tunnel Head (%)	
			1987	1992
12-3	43+00*	500 NW	67	63**
11-3	45+00	0 (145' above)	63	--
10-3	46+00	300 SW	75	66
9-3	55+00	400 NE	41	--
7-3	59+00	375 SW	44	39
8-3	60+00	310 NE	44	36
6-3	69+00	500 NE	29	--

* Shaft Sta. 38+00±
** 1991 data

5.2 Drainage

In late January 1986, piezometers installed south of the tunnels detected pressures rising as much as 700 feet (218 m) in a one week period. The relief wells installed in this area have reduced water pressures in the area. Many piezometers recorded lower pressures after drilling several holes. Within two years, the pressures in the area returned to the levels before the rise and have continued to decrease since.

Piezometers near the ends of the steel liners generally register the highest pressures in the penstock area. Table 2 summarizes the maximum pressures recorded in the past years. Except for piezometer 401, the pressure conditions generally have improved. In the area of piezometer 401, winter pressures rose to about the maximum levels recorded in 1986. However, once this level was reached, the pressures did not remain at that level for long before the general decline began. The maximum pressures may have opened joints leading to the nearby drain holes thereby gradually relieving pressures. This phenomenon may also occur in other areas along the penstock drain curtain. Drains along this curtain are spaced about 7.5 feet apart. This close spacing is offering a excellent defense against the migration of high pressures.

Table 2

Piezometric Pressures Near the Ends of Steel Liners
(Equivalent Water Surface Elevation)
(Maximum Internal Tunnel Pressure at El.3320)
(Lower Reservoir El. 2118)

Piez.	Winter 1985/86 Max.	Winter 1986/87 Max.	Winter 1987/88 Max.	Winter 1989/90 Max.	Winter 1991/92 Max.
401	2808	2809	2810	2741	2795
402	3077	2925	2519	2271	2282
403	2749	2568	2258	2341	2205
404	3019	2964	3028	2773	2732
405	2857	2677	2671	2741	2699
419	3008	2522	2292	2402	2265
420	2876	2510	2420	2262	2232

Down hole backpressures as high as 550 psi have been measured in drains 20 feet from penstock concrete. Minimal evidence of rock deterioration, caused by the high gradient from the tunnels to the adjacent drain holes, has been observed. However, two drain holes located within 13 feet of the concrete penstocks had experienced substantial seepage increase and were grouted. No similar flow increases were observed in adjacent drain holes.

Piezometer data in early 1986 and 1987 indicated that high pressures were migrating downstream along the destressed zone of rock surrounding the penstocks. Drain holes were drilled into the destressed zone to intercept this migration. These drain holes helped control the migration detected by several piezometers downstream of the penstock drain curtain.

Addition holes were drilled toward the penstock crown until concrete was retrieved. The majority of these holes intercepted low seepage volumes (<1gpm) under pressures as high as 510psi. These conditions provide an environment for calcification. Even though they were regularly cleaned and high pressure flushed many have calcified and plugged. There is no plan to replace them since the tunnel and penstock performance continues to be satisfactory.

A drain curtain is located several hundred feet downstream of the concrete penstocks. This curtain, of 112 holes, intercepts less than 7 gpm (0.4 l/s) of seepage. Most seepage comes from holes close to the destressed zones of the penstocks. Piezometers in the area do not detect pressure migration toward the powerhouse.

Upward pressure migration control is supplemented by near horizontal umbrella drain holes drilled from the zig-zag drain tunnels above the penstocks. The hole lengths vary from 185 to 285 feet (56 to 87 m). The horizontal curtain roughly covers the three bifurcations and six concrete penstocks. Many of these holes are flowing, indicating that they are also effective.

6 SEASONAL EFFECTS ON PRESSURES AND FLOWS

There are seasonal patterns for pressures, seepage flows, and displacements. The patterns are more distinct in the penstock area than the power tunnel area. Piezometer readings near the ends of the steel liners, penstock drain tunnel flow, and the total tunnel leakage generally reach their highest level in the winter months lowest in the summer months. Figure 5 illustrates the yearly cycles for piezometric pressures, seepage flows, and displacements. For comparison, the tunnel water temperature is also plotted on an inverted scale.

Tunnel water temperature seems to be the major factor in the seasonal variation. During the fall season when the tunnel water temperature starts to cool, the concrete tunnel lining contracts and cracks in the lining widen. This widening has been confirmed by a remotely operated vehicle equipped with a video camera and recorder. As tunnel cracks widen, the head loss for water seeping across the lining decreases and flow increases.

The maximum winter pressure increase occurs in the concrete penstock area where the grouting pressure was the lowest. The piezometric pressure can increase by about 500 feet (156 m) from summer to winter (Figure 4). Figure 4 also presents the penstock drain tunnel seepage flow. The total flow increases by 800 gpm (50 l/s) from summer to winter.

There are several displacement devices in the powerhouse backslope area which are accurate to 0.02 inch (0.5 mm). These instruments, which include multiple borehole extensometers (MPBX), plumb lines, and span gauges (over existing cracks in the penstock drain tunnels), show recurring yearly cycles. Figure 4 presents graphs for MPBX 364.

In the Lower Power Tunnel area the maximum winter pressure rise is much less than that of the penstock area. Figure 4 presents a plot of piezometer PT4NW3 in the Johnston City Hollow area. The graph indicates a long-term downward trend since February 1986. The small winter rise of 90 feet (30 m) indicates that the high pressure grouting has been effective in producing a head loss zone. There is evidence of a slight time lag between those piezometers located in close proximity to the tunnel (PDVW401) and those located at a distance from the tunnel (PT4NW3).

7 CONCLUSIONS

7.1 The tunnel and penstock performance since December 1985 has been satisfactory.

7.2 The high pressure grouting of the Lower Power Tunnel lining is effective in reducing tunnel leakage and piezometric pressures in the mountain mass. The high pressure mound is contained deep in the mountain.

7.3 The installed drainage is effective in decreasing the general porewater pressures in the hollows near the tunnel route and near the ends of the steel liners. The drainage successfully controls pressure migration upwards and towards the powerhouse backslope.

7.4 A regular drain maintenance program successfully maintains drain effectiveness.

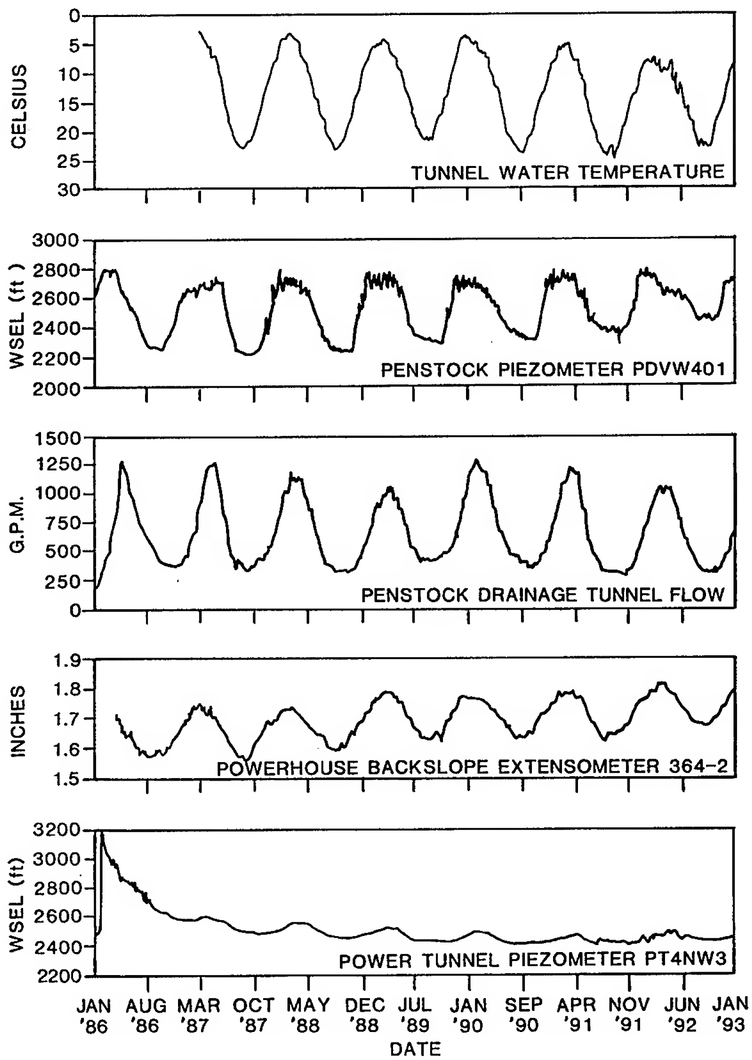

FIGURE 4. PENSTOCK, POWER CORRIDOR, AND POWERHOUSE BACKSLOPE INSTRUMENTS

7.5 Instrumentation data indicates that the highest porewater pressure, flow rates, and rock displacements generally correlate with the coolest tunnel water temperature of the year. Many instruments have recorded data with yearly recurring patterns.

7.6 Minimal rock deterioration in the tunnels and penstocks has been detected.

8 ACKNOWLEDGEMENTS

The authors wish to express their appreciation to the management and staff of Virginia Power, especially Mr. R. W. Cartwright (Vice President, Fossil and Hydro), Dr. J. A. Taylor (Station Manager), Mr. J. M. Daly (Manager, Mechanical Engineering), and Mr. D. I. Young (Project Engineer), for their support and cooperation.

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Finally, special recognition is extended to Mr. A. Zagars, Consultant; Mr. W. J. Bogdovitz, President; and Mr. John Scoville, Chairman, Harza Engineering Company.

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THORNAPPLE HYDRO UNDERSEEPAGE CORRECTION

Richard M. Rudolph, P.E.,[1] John E. Quist, P.E., M. ASCE[2]

ABSTRACT

The Thornapple Hydro Project is a low head hydro owned and operated by Northern States Power Company (NSP) in northwest Wisconsin. Thornapple is constructed on granular fill and native soils. Construction activities on adjacent structures revealed the presence of a potential underseepage problem beneath the powerhouse forebay. Site investigations confirmed the earlier suspicions. Seepage modeling reproduced the field observations. Subsequent to the field data collection and computer modeling, a staged construction project was implemented to correct the situation.

Because of concerns over the potential for loss of structural support and resulting settlement, a comprehensive grouting program was developed. These activities preceded installation of a steel sheet pile cutoff that also served as the construction cofferdam.

Close coordination between construction personnel, the owner, and engineer allowed the project to overcome difficulties encountered during construction while maintaining the tight construction schedule. The project was successfully completed before the deadline and below budget.

PROJECT DESCRIPTION

Northern States Power Company's (NSP) Thornapple Wisconsin Hydro Facility was originally constructed as a grinder mill in about 1909. The existing powerhouse was constructed downstream of the grinder forebay in the 1920s. The six bays of the original grinder bays direct water to the two turbines in the powerhouse downstream. The forebay has been maintained; however, much of the original facility remains. The facility is built of rockfill, timber and timber

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[1]Richard M. Rudolph, Administrator Hydro Operations, Northern States Power Company, 100 North Barstow Street, Eau Claire, Wisconsin 55701

[2]John E. Quist, Project Manager, Barr Engineering Co., 8300 Norman Center Drive, Bloomington, Minnesota 55437

piles, steel sheet piles, concrete, and masonry. Structures are founded on loose to medium sand and gravel fill.

The project goal was to increase the underseepage length, reduce piping potential, and reduce underseepage beneath the powerhouse forebay. The underlying soils are loose near the structures and dense near the bedrock, with blow counts ranging from 1 to 55 in the top layer of the granular fill, and from 3 to more than 100 in the underlying granular till layer. Figure 1, a plan view of the project, shows the primary project features and locations of the soil borings and piezometers.

PRELIMINARY INVESTIGATION AND CONCERNS

Soil borings and piezometers were installed in 1989 to identify soil types, properties, and phreatic levels. Figure 2 is a cross section of the forebay and powerhouse that shows the foundation soils, previous and revised flow paths, sheetpile cutoff, and grouting locations. The piezometers better defined uplift pressures and flow paths, and allowed establishing a monitoring program. A foundation seepage model was developed from the information gathered to approximate existing conditions, evaluate repairs, and identify construction conditions. Figure 3 presents the blow counts and phreatic levels determined from the soil boring program. The model predicted with a high degree of accuracy the conditions measured from the program. Evaluations indicated uplift and soil foundation pressures would be reduced after repairs were completed. Possible reduced foundation support required considerable attention to assure structural stability.

Thornapple is a low head facility that has consistent summer flows. Plant outages during summer construction had to be kept to a minimum, so planned electrical upgrades within the powerhouse were coordinated with the seepage correction. The plant was required to be operational by December 1 to allow testing of the new electrical equipment.

Thornapple is being relicensed and a fish mortality study was underway, so construction in the river could not begin until after July 15. On the other hand. the plant had to be available before winter to accommodate electrical testing. Therefore construction had to be completed between July 15 and December 1.

The short construction time required close coordination between Northern States Power Company (NSP) (engineering and operations), NSP special construction, and Barr Engineering Co. NSP special construction crews constructed the project. Specific work tasks and schedules are described in the following paragraphs.

WORK TASKS AND MILESTONES

Primary construction tasks included:

1. Mobilization of workers, equipment and materials to the site;
2. Foundation penetration grouting of the powerhouse forebay;
3. Installation of steel sheet piling cofferdam/seepage cutoff;
4. Contact grouting beneath the forebay apron slabs;
5. Inspection and evaluation of necessary repairs after dewatering (schedule and budgets were adjusted accordingly);

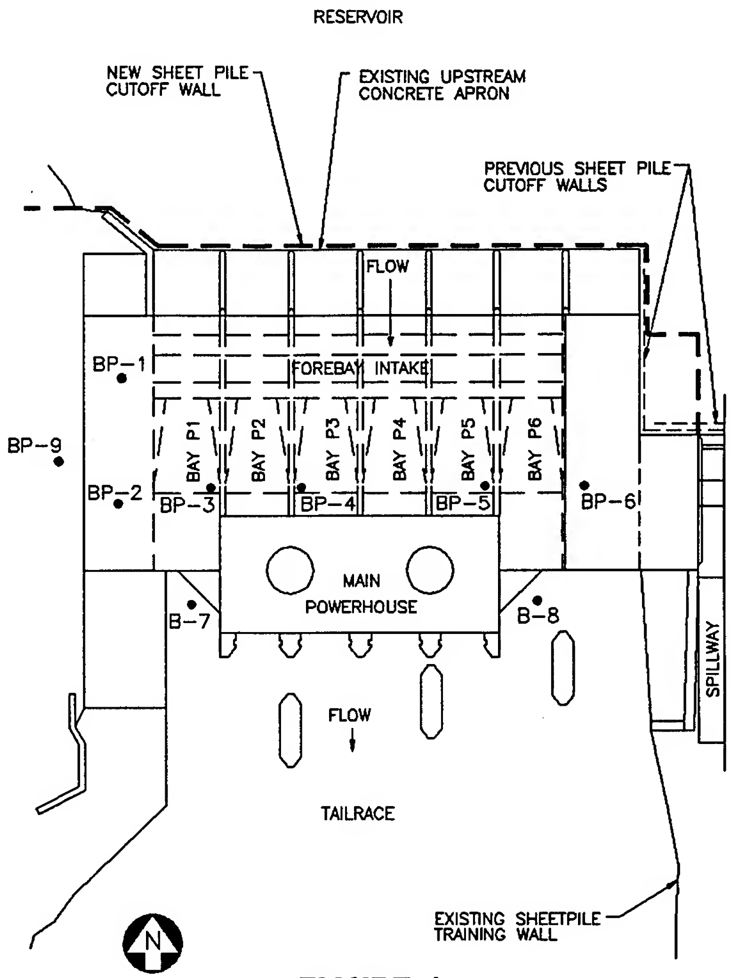

FIGURE 1
SITE PLAN
Thornapple Hydro
Underseepage Correction

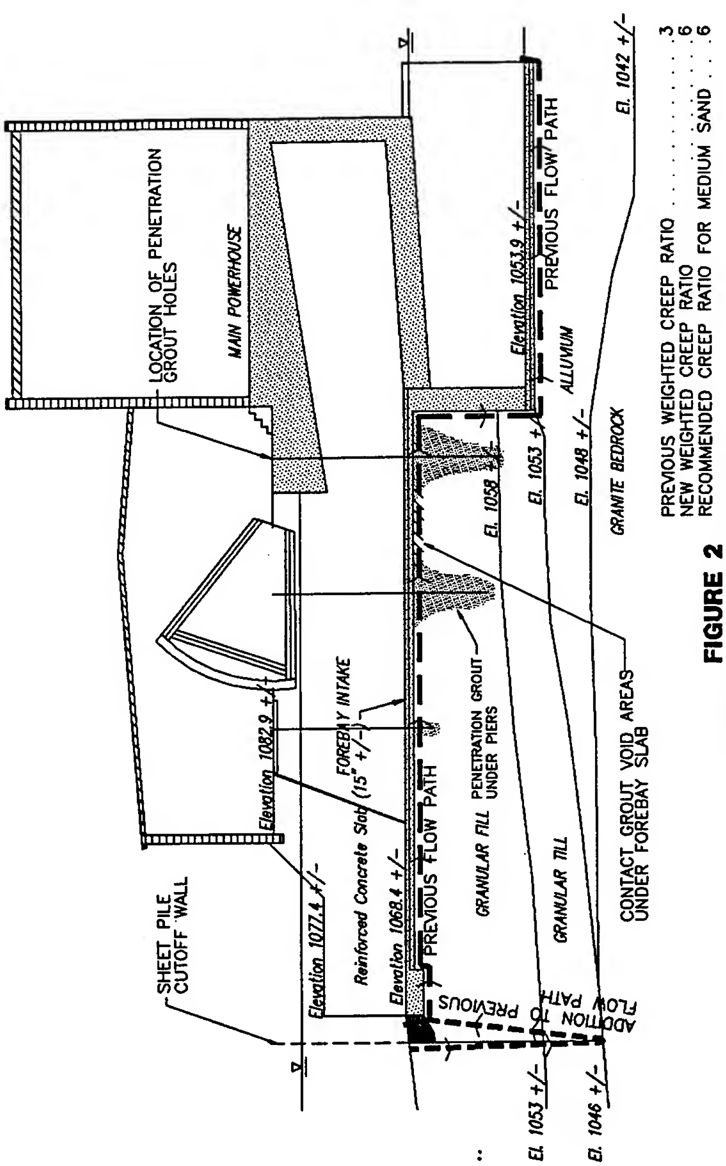

FIGURE 2
POWERHOUSE SECTION
Thornapple Hydro
Underseepage Correction

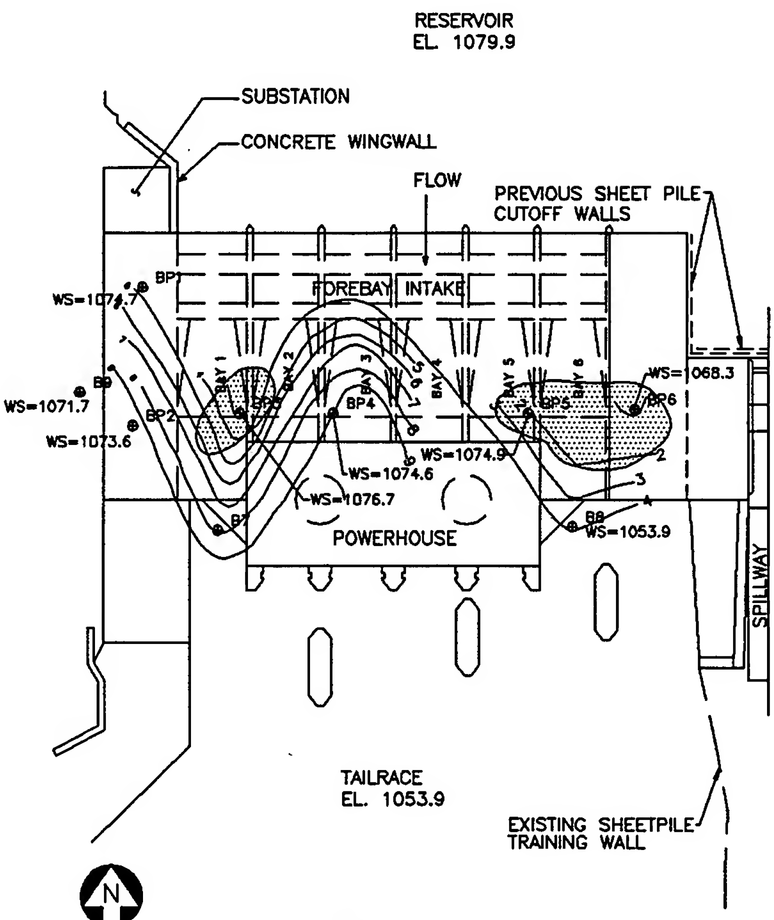

FIGURE 3
BLOW COUNT AND PIEZOMETER DATA
Thornapple Hydro
Underseepage Correction

6. Reconstruction of the forebay piers, slabs, and operator bridges;
7. Refurbishing of the steel trash racks and other miscellaneous structures;
8. Removal of cofferdam, demobilization of workers, equipment, and materials.

MOBILIZATION

Mobilization included planning resources to fulfill the project schedule. Set up of barges, cranes, drill rigs, and grouting equipment as well as grouting materials and sheet piling was accomplished during mobilization.

FOUNDATION GROUTING

Penetration grouting and contact grouting were performed at Thornapple. The penetration grouting was performed prior to installation of the sheet pile cutoff, and contact grouting was performed after pile installation.

Penetration Grouting

To minimize settlement, penetration grouting beneath the primary load carrying components was completed prior to installation of the seepage cutoff. The forebay is enclosed and unheated, which allowed the grouting program to commence in the winter. The grouting was undertaken in two phases (penetration and contact), with penetration being performed first. Penetration grouting was completed on a regular pattern in the areas of identified loose soils beneath the forebay piers. Secondary grouting was completed in and around these areas based on grout takes and other conditions encountered. Robert L. Whartman, of Barr Engineering Co., developed the grouting program and directed field modifications. In addition, careful evaluation of the field observations, and previous experience allowed appropriate modifications to be enacted to accomplish the desired results. Close monitoring of the grout consistency, pressure, and volume placed was the key to success.

Penetration grouting was performed through a 1-inch pipe with a disposable drive point driven into the foundation with a hydraulic hammer until refusal. The pipe was then retrieved and grouting performed at 0.5 to 1.0 foot stages. The grout mix consisted of water, a dispersing agent, and Microfine cement mixed at an average water/cement ratio (by volume) of 6:1. The grouting plan called for each stage to be grouted at a pressure of 20 PSI until locked in pressure of 20 PSI was attained without any take. Although the foundation response caused some variations in grout takes and pressures, the plan was followed routinely on all holes. Grouting on all holes was terminated when the seal around the grout pipe was lost. Twenty-four holes were drilled through concrete totaling approximately 530 feet for the penetration grouting. Approximately 950 gallons of grout were placed during the penetration grouting phase.

Contact Grouting

Contact grouting was performed after the sheet pile cutoff was installed and the forebay dewatered. Contact grouting was performed to fill voids between the concrete structures and the foundation soils. Generally, the grout was placed at a thicker consistency than penetration

grouting. Contact grouting was performed by sealing a grout pipe with an inflatable packer set above the base of the concrete. The grout consisted of a mixture of water and Portland Cement with an average water/cement ratio (by volume) of 0.8:1. The grout plan called for each hole to be grouted until a locked in designated pressure could be maintained. The pressure designated for the piers was 20 PSI and for the floor slab was 10 PSI. Although the foundation response caused some variations in final pressures, the plan was followed routinely on all holes. Approximately 4,000 gallons of grout were placed during the contact grouting phase.

The grouting was completed successfully and no settlement of the structures or other evidence of overstress were observed. Overall, the final grout quantities were slightly less than predicted. This phase of the construction was completed well ahead of schedule.

SHEET PILE INSTALLATION

Steel sheet piling was installed from the right end of the tainter spillway across the upstream side of two non-overflow bays located to the left of the powerhouse then across the upstream side of the forebay to the right bank. Piles were installed through the granular soils identified during the soil boring program to bedrock below or when refusal was encountered.

Consideration as to the type of cofferdam (cantilevered or braced) was made during the early stages. Evaluation of the options focused on the following:

1. Whether the sheets could be driven to have adequate embedment given the anticipated obstructions;
2. Time required to construct a cantilevered wall is less than for a braced wall;
3. Weight and cost of cantilevered wall piles is more than braced;
4. Construction area is more open for cantilevered wall than braced;

Although obstructions were anticipated, it was not believed that they would prohibit installation of cantilevered piles. NSP construction crews were experienced with pile driving and the construction schedule was short, so the cantilevered pile option was selected. Maximum design unbalanced hydrostatic pressures were expected to be approximately 950 psf. This required the selection of a heavy PZ 35 pile. This pile was well suited to the expected hard driving conditions of boulders and timbers identified during the soil boring program.

Initial piles were placed to overlap the existing upstream cutoff that extends from the spillway to increase the horizontal seepage length and permit grouting between the piles for minimizing unwanted seepage. Obstructions from the remains of an abandoned grinder room foundation were encountered during initial pile placement. As a result, the alignment was modified and the piling was installed in tight driving conditions. Piling was installed across the upstream side of the powerhouse forebay to the right bank and eventually into the bank. Piles were driven through the granular soil. Once the piles were installed and the penetration grouting completed, the forebay could be dewatered. Even though the initial installation was delayed, the entire wall was completed on schedule and the forebay intake was dewatered July 18 when generation was discontinues. Sheet pile installation started in June 1991, and was completed July 18 when generation was discontinued. With the pile-installation completed, the forebay was slowly dewatered with no evidence of settlement, overstress, excessive seepage, or leakage observed. Very little seepage was encountered in the forebay upon dewatering. This small

amount of seepage, due to the effectiveness of the pile cutoff/wall, was easily handled by a 4-inch submersible pump that ran intermittently. Monitoring of the piezometers once the sheet pile cutoff was installed and the forebay dewatered indicated approximate lowering of uplift pressures of from 3.3 feet to 10 feet of head with a median reduction of 6.1 feet. This represented a significant reduction of pressures.

The measured values closely matched the values predicted using Lane's recommendations and those predicted using the computer model. Repairs to the intake and modifications were able to proceed once the piling installation was completed.

FOREBAY MODIFICATIONS

Once the area was dewatered, a thorough inspection and evaluation of the concrete structures was permitted. Generally the structures were in good condition; however, the forebay piers and the operator's bridges were found to be so deteriorated that they required replacement. This work was not originally included in the scope but was added to planned concrete apron slab repairs to provide a comprehensive correction. Figure 2 is a cross section of the typical forebay repair that shows the sheetpile wall, forebay, and powerhouse.

The forebay pier noses were removed for each of the six bays and reconstructed with a steel plated armor. Stoplog slots were incorporated into the new pier noses that will allow forebay dewatering. The existing operator's bridges consisted of one-way concrete slabs that spanned the 15.5-foot distance between piers. These bridges were completely removed and reconstructed. The replacement bridges were constructed using composite construction consisting of steel deck pans and a one-way concrete deck slab.

The existing forebay slab was generally 15 inches thick. The top portions of the approach forebay slabs were removed, beginning beneath the bridges and extending upstream to the steel sheet pile wall. The slab and pier concrete was prepared, reinforcing steel placed, water stops installed and shear studs welded to the piles.

The new concrete overlay was approximately 6 inches thick in the main forebay area with a thickened section near the piles. Two small areas of seepage were encountered during dewatering and surface preparation operations. These areas were controlled during concrete placement by locating steel pipes in these areas to collect the seepage, permit concrete placement, and allow for grouting later.

The steel trash racks and support beams were removed, refurbished, and installed.

COFFERDAM REMOVAL AND DEMOBILIZATION

Once the construction and repairs were completed, the construction area needed to be flooded to equalize pressure on the wall. The piling interlocks were very tight with minimal leakage. Flooding the construction area required pumping. The amount of pumping was complicated due to leakage through the turbine wicket gates. The forebay area was eventually flooded allowing removal of the portion of steel sheet pile wall above the new overlay that served as the cofferdam. Divers were employed to cut the piling off, 13 feet below the water

surface, at the top of the new concrete apron. A total of 1,600 sf of PZ 35s were cut off and removed in less than two days.

Demobilization of the crane and barges was hampered due to high river flows and had to be completed on the opposite bank.

SUMMARY

Grouting began during late winter 1991 and installation of the piling initiated in June of that year. A total of approximately 8,800 sf of piling was installed with approximately 7,200 sf permanently in place. More than 528 cy of concrete was placed in the relatively small construction area. The forebay was completed and ready for service on November 17, 13 days before the deadline. The plant was out of service a total of 122 days as a result of this construction. The total project was completed in timely fashion, incorporating additional (yet necessary) work tasks. Obstacles were encountered, but with effective communication and cooperation between Northern States Power Company personnel and Barr Engineering Co., the project proceeded smoothly. Since completion of the project in 1991, the piezometer levels have remained at levels observed after dewatering. The levels recorded after construction are 1 to 2 feet lower than before construction, which represents a 6 to 40 percent reduction in uplift pressures.

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DEER ISLAND PROJECT: AN EFFLUENT DRIVEN HYDROPLANT

By Allen G. Corwin, Acres International Corporation and
Robert D. Getter, P.E., Metcalf & Eddy, Inc.

ABSTRACT

The Boston Harbor Project, Deer Island Facilities, include a 2 MW hydroelectric generating plant located between the disinfection basin and the nine-mile long outfall tunnel. The Boston Harbor Project is being designed and constructed under the direction of the Massachusetts Water Resources Authority (MWRA) and includes primary and secondary treatment facilities located on Deer Island. The effluent entering the Deer Island treatment facilities is pumped to sufficient head to allow the plant to operate at high tide and maximum design flow. These conditions occur infrequently, which provides the opportunity to recuperate some of the energy required for the pumping operations through installation of generating facilities. The hydroplant has a design flow of 28.3 m^3/sec (1,000 cfs) and an operating head of 4.3 to 11 meters. The addition of the hydroelectric plant has no known environmental impacts and merely replaces a conduit used to pass water from the disinfection basin to the outfall conduit.

The design of the hydroelectric facilities included many unique features. The disinfection basin is designed to accommodate the hydroplant, including an additional channel with launders providing sufficient weir length to spill the full flow when the hydroplant is shut down. The arrangement includes a very small forebay volume to allow rapid diversion of flow to the bypass channel in the event of inadvertent shutdown of the hydroplant to minimize surges in the outfall tunnel. This, along with a long tailrace tunnel, required careful consideration of the stability of the load control system. The disinfection basin is divided with the ability to isolate each half from the other. Due to the arrangement of the launders and the requirement to isolate each half of the disinfection basin, a unique intake arrangement was developed. The intake arrangement is also the controlling feature in the configuration of the hydroplant. High concentrations of chlorides and free chlorine result in a highly corrosive effluent, making corrosion protection of the turbine and gate components of primary importance.

Allen G. Corwin, Project Manager, Acres International Corporation, 140 John James Audubon Parkway, Amherst, NY 14228-1180 (Telephone: 716-689-3737), and Robert D. Getter, P.E., Senior Project Engineer, Metcalf & Eddy, Inc., 30 Harvard Mill Square, Wakefield, MA 01888-4043 (Telephone: 617-246-5200)

This paper describes the arrangement of the facilities and design considerations involved in accommodating these unusual features in the engineering of this project.

INTRODUCTION

The Disinfection Facility, and associated hydroplant which is discussed herein, is one of many elements of the Massachusetts Water Resources Authority's (MWRA) Boston Harbor cleanup. The major focus of the cleanup is the new pure oxygen treatment plant on Deer Island in Boston Harbor. The new plant will provide secondary treatment of wastewater flows from over 2.0 million people in 43 communities in the greater Boston area served by the MWRA.

Metcalf & Eddy developed the concept design for the new plant, performed detailed design for primary and disinfection facilities, and coordinated detailed design by multiple firms for the rest of the plant. Acres International Inc. provided planning and design of the hydro facilities as part of the disinfection facilities design package. The new treatment plant is being completed under a court-mandated design and construction schedule. The new plant includes several construction contracts with phased startups beginning with the first phase of the primary clarifiers in 1994 and ending with the last phase of the secondary plant scheduled for startup in 1999. Portions of the disinfection facilities must be available in 1994 for chlorination of effluent from the new primary plant while discharging to the existing outfall. The remainder of the disinfection facilities, including the hydroplant, must be completed in 1995.

The hydroplant will be located between the disinfection basin and the hydro chute which discharges to the outfall shaft leading to the 15 kilometer long outfall tunnel. Two bevel gear units, each with a generating capacity of about 1,000 kW. The total flow capacity of the plant will be about 28.3 m^3/sec (1,000 cfs). The generating units will have slightly inclined Kaplan turbines with adjustable runner blades and wicket gates. The generators will be synchronous type, mounted on top of the turbine inlet and driven through bevel gear type speed increasers.

This paper will discuss the arrangement and design of the hydroplant, and in particular, will elaborate on the items which are peculiar to incorporating a hydroplant into the waste water treatment facility.

PROJECT DESCRIPTION

As the effluent flows through the wastewater treatment facilities, including the grit removal facilities, the primary and secondary clarifiers and the disinfection facilities to the outfall tunnel, is falls from an average head of 47.76 meters (m)[1] at inlet down to an elevation of El 43.15 m at the downstream end of the disinfection basins. The two identical disinfection basins each are equipped with launders with weirs at El 43.07 m which flow into Effluent Channel No. 1. Effluent Channel No. 1 is the reservoir for the hydroplant and is divided into two halves connected by a normally open gate. The intake to the hydroplant is through the bottom of Effluent Channel No. 1. Effluent not flowing through the hydroplant spills through launders into Effluent Channel No. 2 and from there into the chute to the outfall tunnel (see Figure 1).

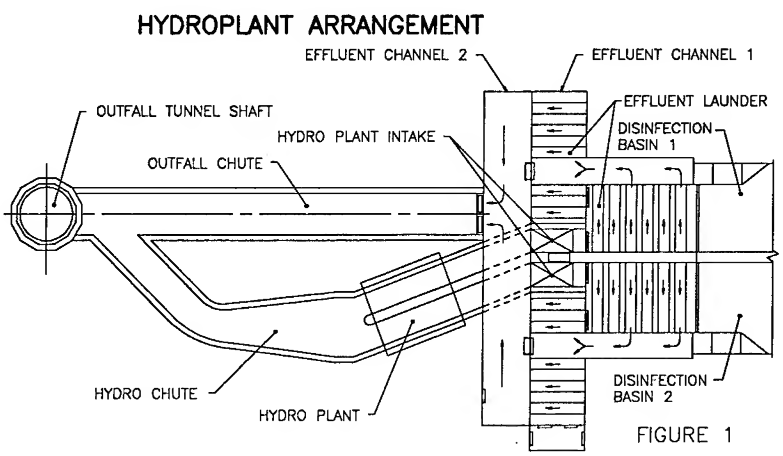

FIGURE 1

PROJECT CONSTRAINTS

The project layout was constrained by several factors:

- The hydro chute, a conduit from the draft tube to the outfall shaft and tunnel, was under construction at the start of the hydroplant design;
- The location of the intake is dictated by disinfection basin process requirements; and
- The length of the space for the hydroplant was restricted to about 26 meters.

Many of these constraints resulted from the fast track approach to the design and construction of the treatment facilities which led to early anticipation of the location and arrangement of the hydroplant. Initially, plans for the hydro station included a double intake from both sides of the Effluent Channel No. 1 leading to a single conduit with several 90° bends. The hydroplant was parallel to the disinfection basin. Based on this arrangement, final design of the hydro chute, the conduit from the hydro station to the outfall tunnel, and the outfall chute, the conduit from Effluent Channel No. 2 to the outfall tunnel, were completed. The location of the hydro chute, which the hydroplant draft tube discharged into, was fixed and could not be changed. Likewise, the location of the north end of the disinfection basin was fixed prior to design of the hydroplant.

SYSTEM HYDRAULICS

Forebay

The forebay of the hydroplant is formed by Disinfection Channel No. 1. The disinfection basin is divided into two equal parts to provide redundancy and capacity for maximum flow conditions. Effluent from each half of the basin flows over weirs

at El 43.06 meters (m)[1] into the launders (troughs) to enter the Effluent Channel No. 1. Effluent Channel No. 1 is also divided and has a normally open gate located between the two halves. Six launders with weirs at El 42.52 m provide over 100 meters of overflow capacity for each half of the disinfection channel. The total surface area of Disinfection Channel No. 1 is about 550 m^2.

Hydroplant Intake

To ensure satisfactory operation of a hydraulic turbine, in particular an axial flow turbine, it is important that the approach flow to the turbine runner is uniform. Thus the hydroplant was fitted into the site at an angle to the disinfection basin in order to eliminate several 90° bends included in the conceptual arrangements.

The intakes to the hydroplant are through two 33.4 m^2 openings in the Effluent Channel No. 1 floor (El 35.66 m) into the hydroplant intake conduits. The intake conduits slope from El 29.57 m down to El 28.43 m.

Isolated operation of each half of the disinfection basin was a design condition. Intake openings from each side of the effluent channel were necessary to permit isolated operation of each half of the basin. Also, it was determined that a single opening from one half of the basin could result in cross flows in the intake and unacceptable flow patterns at the units. Therefore, the powerhouse is arranged so that intakes from each side of Effluent Channel No. 1 would flow through two separate conduits to two separate units (Figures 2 and 3).

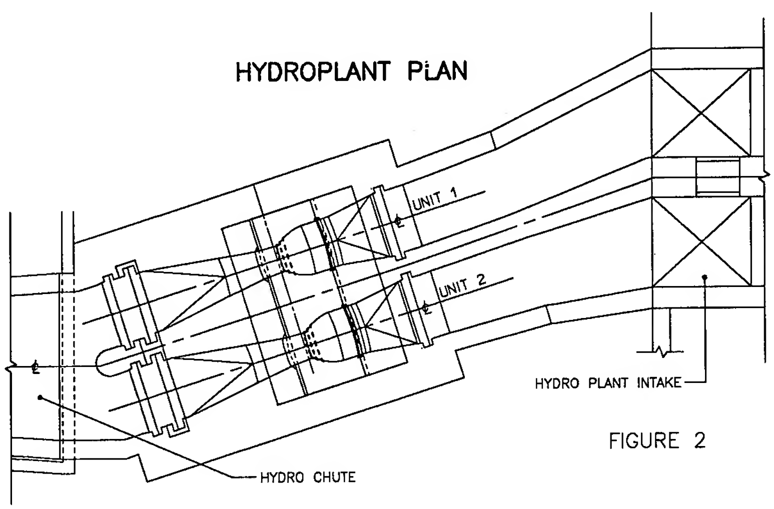

FIGURE 2

[1]Elevations are based on the Metropolitan District Commission Sewer Datum (Mean Sea Level = 32.193 meters).

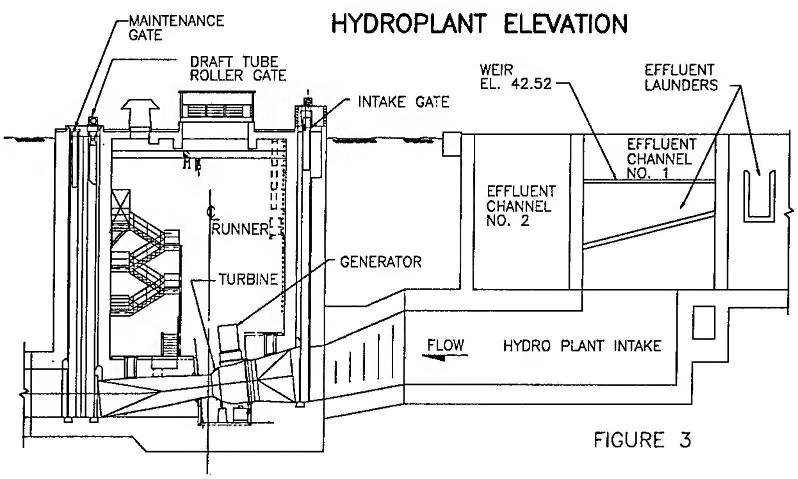

FIGURE 3

Velocities at the intake opening were kept below 1.5 ft/sec to reduce the chance of vortices formation and provide uniform flow at the units. Also, the powerhouse was located as far downstream of the intake as practical to provide a straight length of conduit between the intake and the hydraulic turbine.

Outfall Tunnel

The draft tubes of the two units discharge into a 4.57 m high, 41.45 m long hydro chute. The hydro chute is 7.70 m wide at the draft tube entrance located at El 29.47 m and narrows to 5.18 m wide. The chute has a 45° bend at 18.90 m downstream of the entrance and discharges into the outfall chute at El 27 m, about 7.62 m upstream of the outfall shaft.

The outfall shaft is 9.14 m in diameter and extends from the surface (El 45.72 m) down to 73.15 m below the sea bed (-El 82.6 m). The shaft elbows into the 7.32 m diameter outfall tunnel, 15 kilometers in total length, with a tapered diffuser section about 2 kilometers long. Fifty-five risers extend from the bottom of the tunnel to a diffuser cap that will discharge flow horizontally with a radial distribution.

System Hydraulics

The hydraulic stability of the outfall tunnel was identified as being of critical importance in the design of the hydroplant. The stable flow of effluent through the tunnel assures the continued operation of the tunnel without incursion of sea water. Although the tunnel was designed to operate below the minimum flow level expected through the treatment plant, concern was raised about interruption of flow during normal or emergency shutdown of the hydroplant. Should shutdown of the hydroplant result in extended periods without supply to the tunnel, there would be a transient condition during which there would be the potential to draw sea water back into the tunnel. It was determined that if a positive head were maintained at the outfall shaft at all times, then extensive transients would not pose a problem.

A second consideration was the relatively small "forebay" area of 548 m^2. The effluent level in Effluent Channel No. 1 was to be maintained just below the weir by the automatic adjustment of the wicket gates by the Programmable Logic Controller (PLC) headwater controller. Analysis of the response times showed that slow wicket gate times resulted in unstable headwater level control. Therefore, a relatively fast wicket gate time of four seconds was selected as the best for the stability of effluent levels in Channel No. 1. This time was then checked against the interruption of flow to the outfall shaft. It was determined that substantial flow would be restored over the weirs via the outfall chute to the outfall shaft in about seven seconds. This was found to cause no significant transient waves in the tunnel and provided for satisfactory operation of the hydroplant.

UNIT SELECTION AND ARRANGEMENT

The hydropower facilities design will incorporate two 1,000 kW bevel gear bulb units each with a flow capacity of about 14.15 m^3/sec (320 MGD). The selection of the appropriate plant layout included evaluation of the plant flow, type of unit, and the number of units. The selections were based on financial analyses and other engineering considerations. The financial analysis consisted of two components, the energy analyses and the preliminary cost estimate.

Type of Turbine

The total flow through the Disinfection Facility is expected to vary from a minimum of 12.74 m^3/sec (290 MGD) to a maximum of 55.64 m^3/sec (1270 MGD). The available head of the hydropower facilities ranges from approximately 10.97 m to less than 3.05 m over the ranges of flows and tides. For these heads and flow conditions, an adjustable blade propeller (Kaplan) type turbine is most appropriate. Various Kaplan turbine sizes and configurations were evaluated.

Flow Capacity of Plant

The flow duration curve was used in the evaluation of the flow and in all subsequent energy calculations. To assess the relative economics of various sizes of hydropower facilities, energy analyses were made for maximum hydroplant flows ranging from 22.65 m^3/sec (520 MGD) to 45.3 m^3/sec (1030 MGD).

The results are tabulated below:

Maximum Plant Flow		Annual Generation (MWH)	Incremental Energy (MWH)
m^3/s	(cfs)		
22.65	800	11,610	--
28.32	1,000	12,500	940
33.98	1,200	12,970	420
39.64	1,400	13,120	150
45.31	1,600	13,090	-30

As maximum plant flow increases, the incremental energy becomes less. For the higher flow annual generation actually decreases, because larger turbines would be selected, which when operated at low flows, would have reduced efficiency.

The financial analysis indicated that the two lowest flow options (22.65M^3/sec and 28.32 M^3/sec were most economical. The 28.32 M^3/sec alternative was selected because of significantly increased generation when compared to the 22.65 M^3/sec option. Flow exceed the selected plant flow of 28.32 M^3/sec approximately 20 percent of the time. This is a typical maximum flow for a modern hydroplant.

Unit Configuration

Several schemes were considered for the hydropower facilities.

A financial analysis of each of the schemes was made based on preliminary energy estimates and capital cost estimates for each option. The two-unit installation with bevel gear type bulb unit was selected as an optimum balance between performance and/or reliability and economy.

The single unit bevel gear bulb, the S-type Kaplan turbine, and the open flume vertical Kaplan all had shorter payback periods than the selected option. However, the intake conditions were questionable for a single horizontal axial flow machine, thus eliminating the single bevel gear bulb and the S-type Kaplan from further consideration. The open flume Kaplan had only a marginally better payback period and was not selected because very few of these types of turbines have been built in recent years making equipment procurement and predesign of the powerhouse more difficult. Also, much of the advantage of an open flume intake would be eliminated because of the intake configuration required to support the disinfection process.

In addition to these drawbacks stated for the open flume full Kaplan, the open flume semi-Kaplan turbine, which was also evaluated, may have less precise flow control when compared to designs with wicket gates. This may lead to problems with flow/water level control because of the relatively small surface area of Effluent Channel No. 1 and the necessity to maintain water level constant by matching turbine flow with the Disinfection Facility effluent discharge.

The vertical Kaplan proved to be least attractive financially and also is somewhat unusual in this size unit, increasing the procurement and design problems as the case for the open flume machines.

Our decision was also influenced because the bevel gear bulb units are available as a standard design; therefore, this scheme provides the best opportunity to design the powerhouse prior to purchase of equipment, a requirement due to the schedule and procurement procedures. The other options would likely have required custom-designed turbines. The overall reliability of a two-unit installation was believed to be somewhat better than the single unit plant. Furthermore, the two-unit plant would incorporate more rigid, compact units and would have a speed increaser rating which is more in line with present day experience for hydro turbine applications. The bevel gear bulb turbines are relatively small and can be shipped fully assembled, permitting relatively easy installation.

FACILITIES DESCRIPTION

The turbines will be horizontal (or slightly inclined) axial flow type in a bevel gear bulb configuration. The generator will be driven through a right angle drive speed

increaser located in a pit or "bulb" at the upstream end of the turbine. The generator will be vertical (or sightly inclined) and mounted on top of the turbine.

The turbine will be Kaplan type with both adjustable runner blades and adjustable wicket gates to control the flow through the turbine. The hydraulic design conditions for the turbine and the turbine design parameters are summarized below:

GENERATING UNIT PARAMETERS

Headwater Level:

• Maximum	El 42.75 m
• Normal	El 42.43 m
• Minimum	El 41.22 m

Tailwater Level[1]:

M^3/SEC			
FLOW	LOW TIDE	MEDIUM TIDE	HIGH TIDE
56.63	36.27	39.01	40.54
42.48	34.90	57.30	37.49
28.32	32.92	34.14	35.66
14.15	31.39	32.61	34.14
0.00	30.92	32.16	33.68

[1] Tailwater level is the hydraulic gradient level at the outfall conduit.

Turbine Net Head:

• Maximum (with 12.74 m^3/sec, 290 MGD flow)	11.13 m
• Rated	8.53 m
• Minimum	3.2 m

Turbine:

• Type	Full Kaplan
• Rated Output	1,080 kW
• Runner Diameter	1.45 m[2]
• Speed	350 rpm[2]

[2] Preliminary

Speed Increaser:

• Type	Bevel gear

Generator:

• Type	Synchronous
• Rated Output	1,110 kVA
• Power Factor	0.9
• Voltage	4,160 V
• Speed	900 rpm

The turbine runner and discharge ring will be of stainless steel or aluminum bronze construction to minimize the possibility of cavitation damage. The outer line of the turbine runner is set approximately 1.5 m below minimum downstream water (tailwater) level based on considerations for preventing cavitation damage and the configuration of the disinfection basin and hydro chute. The turbine guide bearing will be oil lubricated. The turbine wicket gates will have self-lubricated bearings.

The right angle drive speed increaser which connects the turbine to the generator will be designed in accordance with recognized standards for a life of at least 40 years, based on anticipated operating conditions. The speed increaser bearings will also be oil lubricated and will incorporate a thrust bearing, designed to transfer the hydraulic thrust from the runner to the turbine foundations.

The turbine wicket gates and runner blades will be operated by hydraulic servomotors with oil supplied by a hydraulic system with electric motor driven pumps, a sump tank and interconnective valves and piping. The wicket gate operating mechanism will have a counterweight to ensure closure of the wicket gates and stopping of the unit in the event of loss of oil pressure to the servomotor. The turbine will be controlled by an electronic positioner which adjusts the opening of the wicket gate and runner blades. The positioner will incorporate an electronic three-dimensional "cam" which adjusts the runner blades depending on the wicket gate position and the head across the turbine so as to maximize the efficiency of the turbine.

The synchronous type generator will each be rated at 1,000 kW and operates at 4,160 volts, 3 phase 60 Hz. Performance guarantees will be provided by the equipment supplier.

Hydroplant Operational Characteristics

Effluent discharges from the disinfection facilities up to the flow capacity of the hydropower facilities will pass through the hydro units for generation of electricity. Control of the hydro units will be with a programmable logic controller (PLC). The PLC will adjust the opening of the turbine wicket gates (through the electronic blade/gate positioner, based on small variations of water level in Effluent Channel No. 1, so as to match the discharge of the hydro units with the disinfection facilities flow. The PLC will also start and stop the second hydro unit on increasing and decreasing flows. The flow at which the second unit starts and stops will be established so as to maximize the efficiency of the hydropower facilities.

When disinfection facilities flows exceed the flow of the generating units, the level in Effluent Channel No. 1 will rise and excess flow will spill over the weirs and discharge through Effluent Channel No. 2 to the outfall conduit and outfall shaft. When the generating units are shut down, all flow from the Disinfection Facility will pass over the weirs and through Effluent Channel No. 2.

CORROSION PROTECTION

The effluent of wastewater treatment facilities typically contain elevated concentrations of chlorides and other corrosive chemicals. For the Deer Island facilities, this condition is exacerbated by the inadvertent leakage of sea water into the sewage collection system elevating the chloride content. The expected effluent water quality is:

• maximum chlorine content	5 mg/l
• maximum chloride content	4,000 mg/l
• maximum sulfides	5 mg/l
• maximum phosphorus	4 mg/l
• maximum nitrogen	30 mg/l
• pH range	6.0 - 6.5

Materials for the critical rotating and the discharge ring are selected to provide maximum protection against corrosion, erosion and cavitation. Nickel aluminum bronze is selected for the runner hub and blades, and 316 stainless steel is used for the discharge ring. The remainder of the turbine exposed to the effluent is to be of fabricated steel, cast steel or nodular cast iron construction.

To protect the ferrous materials against corrosion, high solids epoxy paints are specified. However, it was recognized the painted steel or cast iron would probably not hold up over the long term.

Review of other installations that were installed in rivers or estuaries with relatively high concentration of chlorides revealed that extensive corrosion problems have occurred. Corrosion damage was observed between dissimilar materials such as chromium, nickel, steel and carbon steel. Corrosion at the interface between the discharge ring and the wicket gate assembly, the runner hub and carbon steel shaft, the runner hub and carbon steel bolts has been reported to be severe in some cases requiring replacement turbine components including turbine cases and shafts. Also, areas which were not coated or where the coating was rubbed off, such as ends of wicket gates, mating bolted surfaces, bolt heads, and similar areas, had been attacked. Therefore, alternate and/or additional methods of corrosion protection were determined to be necessary.

Cathodic Protection System

To protect the carbon steel components of Deer Island turbines and gate guides, a Cathodic Protection System was specified. Loose electrons are supplied to the effluent through a total of 24 anodes positioned throughout the turbine and the concrete adjacent to the turbine components and gate guides. Reference electrodes are located on or adjacent to the protected surfaces to provide feedback to the automatically controlled rectifier. The rectifier supplies current to the system, and the automatic control system maintains a constant potential between the anode and reference electrode of up to 850 millivolts (MV). It is important not to exceed 850 MV because electrolysis may cause separation of water into its component gases causing an explosive condition inside the water passages.

All anodes and reference electrodes are mounted to be flush with the water passage surface to cause a minimum of disturbance.

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Water Treatment Plant No. 2 Power Facility Project Development

William H. Blair [1]
Paul R. Kneitz [2]

Abstract

The Alameda County Water District is completing a water treatment facility which uses ozone for disinfection. The water supply for this treatment plant is delivered through a hydropower development.

The integration of the hydropower and water treatment projects, the mitigation of environmental impacts and the requirements of agencies having jurisdiction lead to a final project configuration somewhat different from the best scheme originally conceived.

The changes to the original best scheme and the factors causing these changes are described. Final project configuration includes: four 250 KW and two 125 KW generating units, two bypass valves and electrical equipment in a reinforced concrete structure, a connection to the serving aqueduct and a connecting pipeline.

Introduction

The final configuration of this project is described above in the abstract. The purpose of this paper is to describe the process of project development and how nontechnical factors--environmental, permits, utility coordination, schedule and contracting--influenced project design and configuration.

This project is unique: the marriage of two projects (hydropower and advanced water treatment) on a limited site. Two

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[1] Design Manager, Alameda County Water District, P.O. Box 5110, Fremont, CA 94537-5110
[2] Vice President - Hydro & Water Resources, Tudor Engineering Company, 1800 Harrison Street, Oakland, CA 94612

water for this project is delivered to the District at an elevation well above (about 92 meters (300 feet)) the site. This head is dissipated as the hydropower facility generates electric power, much of which is used to produce the ozone used in the treatment process.

The Alameda County Water District is a California special district established in 1914 which provides water service to a 1990 population of 263,860 in the cities of Fremont, Newark and Union City in the San Francisco bay area. The service area is about 259 square kilometers (98 square miles); there are about 70,000 customer accounts (75 percent residential); in a normal water year the District delivers about 63,450,000 cubic meters (50,000 acre feet) of water. In addition the District actively manages and protects the local Niles Cone Groundwater Basin.

The District has three sources of water supply: local watershed runoff (15 percent); the California State Water Project (60 percent) and the San Francisco Water Department/Hetch Hetchy Project (25 percent). A planning and facilities study completed in 1986 forecast future water demands and analyzed in detail several alternatives for meeting these demands using the District's three sources of water. Conjunctive use of the District's surface water and groundwater supplies was the only alternative that met future demands. Therefore it was recommended that the District construct additional surface water treatment capacity, increase groundwater recharge capability and add production wells.

Groundwater recharge and production well projects were started immediately in 1986 as they had relatively short design and construction time lines. Work on the water treatment facility projects was started in 1988 with an expected 3 1/2 year design and construction period.

When design development started in 1988 there were three givens: the site, the ultimate capacity of the water treatment facility and the use of ozone in the water treatment process.

The site was identified through a site selection study performed in 1981. Candidate sites (vacant land) were compared by means of a selection matrix. Power use or power production was the most significant economic factor in the comparison of the sites. The site selection study recommended the Mission/I-680 site. Acquisition of that site was completed in 1985.

The District's 1987-1992 Capital Improvement Program (CIP) justified an initial water treatment plant capacity of 36,800 liters per minute (14 MGD), with provision for a future 18,400 liters per minute (7 MGD) expansion, for an ultimate total of 55,200 liters per minute (21 MGD).

Historically the primary disinfectant of choice in the United States has been chlorine; in Europe it has been ozone. Trihalomethanes (THMs) are formed by the reaction of chlorine with organic precursors. In 1979 these disinfection byproducts were regulated to a low level by the U. S. Environmental Protection Agency (EPA). In 1988 it was expected that the USEPA would regulate THMs to a substantially lower level. Ozone, a proven water treatment technology, would make compliance with expected future regulation possible.

Consultant Selection and Use

The District uses consultants when projects are large and/or require specialized expertise. The selection process involves identifying firms with the appropriate experience, then requesting proposals based on a District outline of the scope of the assignment. Individual preproposal meetings are held with the consultants to explain District requirements in more detail and most important to allow the consultants to ask questions. After proposals are submitted the District reviews them and prepares a preliminary selection matrix in which the various elements of each consultant's proposal are ranked against each other. Questions concerning the proposal are also developed.

The top ranked consultants are then invited back and given an hour to introduce their project team, present their plan for doing the work and answer the District's questions concerning their proposal.

The final selection is based on an evaluation of all the data received--the proposal and the presentation. This evaluation is done using a matrix similar to that used to evaluate the proposals.

Separate consultants were selected for the water treatment facility and the head breaking (hydropower) facility and its supply pipeline.

The District has found that it is cost effective to work closely with its consultants. Regular progress meetings are held to review progress, consider alternatives and make decisions. These meetings minimize surprises and the necessity for rework. Design is usually done in two stages. The first stage is problem definition, consideration of alternatives, development of the design and design criteria to a level of completion that allows the design to be frozen. Agencies having permit authority are identified and final design and construction schedules are established.

The second stage is the final design itself--the preparation of drawings and specifications needed to obtain competitive bids for construction.

Development of the Best Scheme

Preliminary work done by the District identified some of the issues to be considered in determining the best scheme of development. For the head breaking facility these included: type and number of turbines, type of head breaking valve, type of generators and utility coordination.

For the supply pipeline these included: location of the new connection to the Department of Water Resources (DWR) South Bay Aqueduct (SBA), pipeline alignment and diameter and the location and type of the pipeline crossing of the eight lane freeway separating the project site from the SBA.

The District's direction to the consultant was to develop a broad range of alternatives and at least initially disregard institutional obstacles.

The best scheme of development for the hydropower facility included three fixed geometry turbines driving induction generators with connection to the power utility at the site. At that time the District had a wheeling proposal under consideration by the utility.

The best scheme of development for the supply pipeline was not as clear cut. The shortest alignments required connection to the SBA where there was no unallocated capacity. Adverse effects on the downstream agency proved to be an institutional obstacle which could not be overcome. Therefore the connection to the SBA was located where there was unallocated capacity. This required a somewhat longer pipeline. A major feature of this alignment is a 250 meter (812 foot) long diagonal cased crossing of the I-680 freeway. For the selected alignment the most economic diameter was 84 cm (33 inches). The more standard pipe size of 91 cm (36 inches) was the final selection.

The consultant identified the agencies having jurisdiction and the permits and agreements that would be needed and developed a detailed schedule for permits, final design and construction.

We were on schedule, as it had taken only two months to develop the best scheme. However, during this same time period the Environmental Impact Report (EIR) for the District's 1987-1992 Capital Improvement Program was being prepared.

Environmental

The environmental process makes early, full disclosure about projects. The comments received and discussion of your project may not be what you expected to hear, but the affirmative response to the public concerns makes projects possible.

Work on the program EIR for the District's 1987-1992 CIP was started in late 1987. The water treatment facility and hydropower facility were each projects included in the CIP. By April 1988 we were ready to hold an informational meeting for the residents living near the project site. The neighbors turned out in force and stated substantial concerns for aesthetics, sound pollution, light pollution, traffic safety, solids drying beds and hazardous material safety. These concerns were repeated at a public hearing held in June 1988 on the draft EIR.

The District's Board then directed that additional engineering studies be performed to address the public's concerns. It took about seven months to complete these studies, during which time the design report was completed to a level that would support permit applications.

Design Report

As the District began to obtain more information about the requirements of the agencies having jurisdiction over the development, it was necessary to modify the best scheme.

The power utility declined to wheel our excess power production to our wellfield load center; the District learned of the number and timing of design reviews by DWR; and operational considerations required that two half size turbines at 9,200 liters per minute (3.5 MGD) each be added to allow a greater selection of water treatment plant operating rates.

The decision of the power utility was a disappointment but led to a study of a District owned electrical interconnection from the hydropower facility to the existing Mission San Jose Water Treatment Plant (MSJWTP). To serve present and future electrical loads there, a 4,160 volt electrical interconnection to the MSJWTP was added to the best scheme.

The DWR design review procedure was accommodated by making the SBA connection a separate design and construction package with an early start date.

Before the final EIR was considered by our Board in April 1989, another informational meeting was held with the neighborhood to show how the project plans had been modified in response to their concerns. This second meeting one year later was much less eventful than the first. Board approval of the design report (and authorization of final design) took place shortly after the EIR was certified.

Site Related Decisions

The environmental process delayed our original schedule about a year. Because of neighborhood concerns about traffic, aesthetics and the construction process, it was decided to do

preparatory work in advance of plant construction. This work included street widening, extension of utilities onto the site and an architecturally pleasing frontage wall with landscaping. It was also decided that there should be only one contractor on site for the two projects. Therefore the drawings and specifications for the hydropower and water treatment facilities, although prepared by different consultants, had to be well coordinated.

This coordination was successful because of well established lines of demarcation between the two facilities and because the District actively managed the coordination process.

Permitting and Final Design

The requirements of the agencies having jurisdiction need to be incorporated in the design of any facility. However, usually a near final design is required before an agency will make a review or issue a permit. Agency requirements need to be determined early so that the design can be developed without excessive rework.

Our consultant identified the following list of agencies and permits needed for this project:

Agency	Permit
Federal Energy Regulatory Commission (FERC)	Qualifying Facility (QF) certification
FERC	Small conduit exemption
DWR	Turnout contract Encroachment permit
Caltrans	Encroachment permit
Pacific Gas & Electric (PG&E)	Power sale agreement Parallel operation agreement

Our QF application was filed in January 1989 and certification was received in April 1989.

The agency consultation required by FERC started early in 1989 as soon as the environmental studies were complete. Our application for a small conduit exemption was accepted for filing in October 1989; the exemption was received in March 1990.

Final design started in June 1989 with meetings with DWR and Caltrans to explain our project and obtain detailed agency requirements. These meetings helped define the scope of the

permit application packages. We also found that there was no fatal flaw in our overall plan.

By this time our project milestones had been rescheduled: process design complete by the end of 1989, final design complete by November 1990 and construction complete by May 1993.

The long lead time items on the design schedule were: DWR design reviews of the SBA connection, right of way acquisition for the supply pipeline, the Caltrans encroachment for the pipeline crossing of I-680, and the many technical and procedural steps described in the PG&E Power Producers Interconnection Handbook.

The hydropower facility schedule was the same as that for the water treatment facility. The schedule for the SBA connection and the supply pipeline was driven by the need for water for operational testing of the water treatment facility by the end of 1992.

For the SBA connection and the supply pipeline final design resulted in only minor changes to the best scheme. Two gravity flow pipelines and a bank of conduits serving the MSJ Water Treatment Plant were added. These additions required an increase in the diameter of the I-680 crossing casing.

During final design the best scheme for the hydropower facility underwent relatively more changes than the best scheme for the supply pipeline. Change was caused by a better understanding of the economics of power purchase and sale, the physical integration of the hydropower and water treatment facility and by the need to mitigate environmental impacts.

For electric energy in 1988, the economic factors were: $0.087 per kilowatt hour (KWhr) to buy, $0.027 per KWhr to sell and $2.35 per kilowatt of demand for standby.

The production of ozone is energy intensive. This power demand was compared with generation. There was surplus generation available for sale or use for all combinations of plant flow rate and ozone application rate.

As there was always a surplus of generation, the ozone production load could be served entirely by the hydropower facility. To reduce standby charges this load is not served by the utility.

Although simple, sale of the surplus generation with subsequent repurchase for other plant electrical loads was not economically attractive. Internal use of the surplus was complicated by seasonal variation in the amount available. Another complicating factor was the need to serve all plant loads with an engine generator on utility failure.

The adopted design established two electrical bus systems - one for power sale to the utility (sell bus), the other for power purchases from the utility (buy bus). In plant electrical loads were divided into three blocks which can be connected to either bus. The engine generator is connected to the sell bus. In the event of utility failure the engine generator starts automatically and automatic transfer switches connect all loads to the sell bus. Provision was made for serving future ozone loads at MSJWTP from the sell bus through a District owned 12 KV interconnection.

The layout and design of the hydropower facility was modified to control the environmental impact of sound pollution to an acceptable level off site. The walls and ceiling of the turbine room received an acoustical treatment. The engine generator was placed in an acoustic housing in a separate room behind a wall of water, the tailrace structure.

Contract Provisions

Because of the dollar value of the contract ($30,000,000) and the 26 month construction period the District made some changes to its usual contracting practices.

A cost loaded CPM schedule updated monthly was required. Monthly earnings are determined from the CPM schedule updates.

The 26 month construction period includes a 60 calendar day allowance for delays beyond the control of the contractor. The allowance has acquired a name: "the bank." The contractor is required to promptly notify the District of all delays. If the District determines that a delay was beyond the control of the contractor, these delay days are deducted from the bank. Delays do not extend the final completion date until the 60 day allowance is used and then only if the delayed activity is on the critical path.

There is an early completion incentive of $5,000 per day (limited to 120 calendar days) as well as liquidated damages of the same amount for late completion.

The project was bid on a lump sum basis with an established price for mobilization. A unit price for extended overhead also entered into the comparison of proposals.

A common contracting problem is determining completion on an objective basis. To help solve this problem, each technical specification section was reorganized to group testing requirements in a separate part. This new part specifies the tests, required results and responsibilities--who performs, who witnesses, who pays.

The contractor was required to retain an independent Electrical Testing Agency (ETA) and a Special Test Engineer

(STE). The ETA is required to perform short circuit and coordination studies and test and verify the electrical installation down through the 480 volt level. Deficiencies have been discovered but there has been enough time to make corrections without affecting the completion schedule.

The STE is required to supervise the overall start up of the hydropower facility. As of March 1993 this work had not been done. However, an STE was specified and used on another District power project with very satisfactory results.

Construction

Construction has been much less eventful than the design. Subsurface conditions were as anticipated, with competent sandstone found under the hydropower building and large pieces of rock found in the highway embankment during the tunneling operation. The early construction of the site improvements was a good investment. Water and power on site enabled the contractor to start work sooner. The frontage wall has served its purpose of screening construction activities from view. The contracting provisions unique to this project have been beneficial to project construction management.

Manuals and Training

Complete operating manuals for the hydropower facility have been prepared in advance of start up. These have been used as the basis for class room training of the operating staff.

An innovation for the District was the development of a "quick reference card" for the operating staff. One side of this card contains essential operating instructions. The other lists the proper response to alarm messages.

Observations and Conclusions

- It will take longer than you expect.
- Consultant selection is the most important project task.
- Time for permit and right of way acquisition needs to be included in the project schedule.
- The requirements of agencies having jurisdiction will be an important factor in project design.
- Thoroughly understand the rules and rate schedules of the serving electric utility.

125 KW Small Hydro Power Plant
For Escuela El Sembrador
Catacamas, Honduras, C.A.

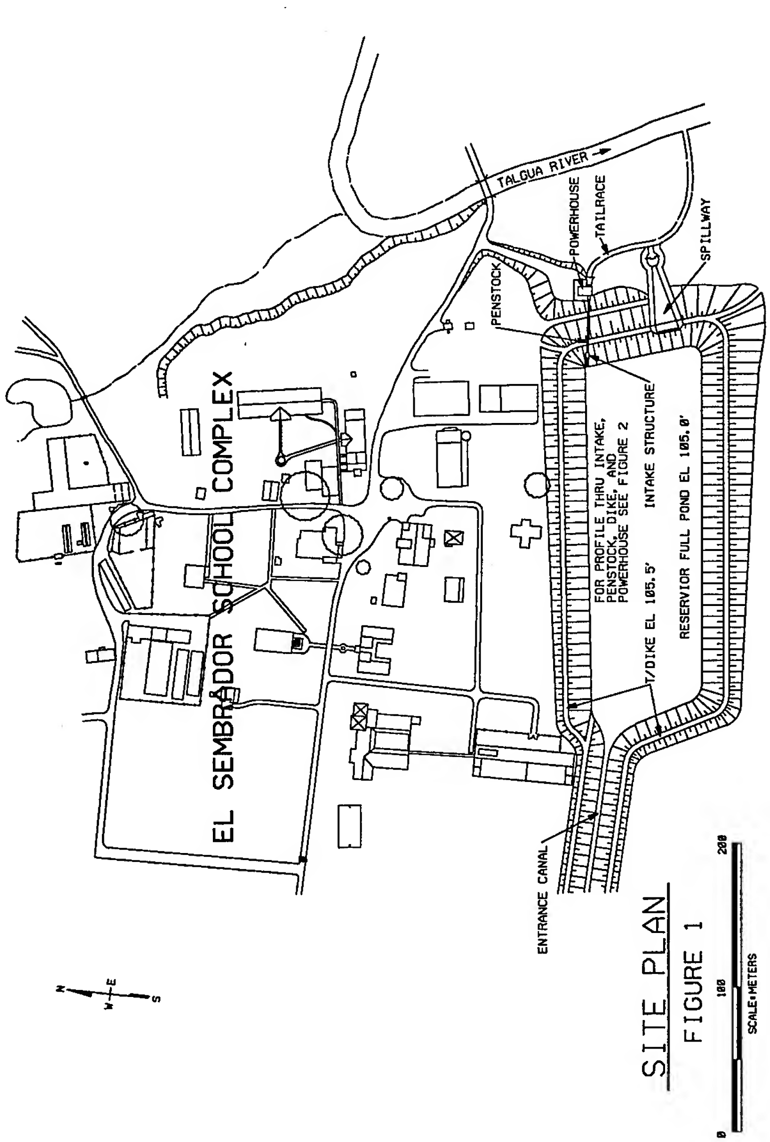
Edward D. Campbell, P.E., Life Member ASCE [1]; William D. Wright, P.E. Member IEEE[2]; and Hugh G. McKay, III, P.E., Fellow ASCE [3]

Abstract

Small hydropower in third world countries is a valuable asset as a replacement for diesel generated electric power. The incentive for this project was that it could be developed and constructed in conjunction with an existing irrigation project, and supply electrical power for general use and for deep water well pumps. The scope of the project included not only generation of the electric power, but a complete electrical distribution system for a vocational training school compound. The 32 buildings that comprise the El Sembrador agricultural and vocational training school is spread out over 40 hectares which required the use of a 13.8 KV over head powerline grid. See Figure 1. From specific locations along the powerline, pole transformers feed energy to groups of buildings through low voltage distribution panels. The hydroplant itself is a 125 KW crossflow turbine with a three phase 480 VAC synchronous generator. The water supply is from a 4 hectare reservoir created by an irrigation water canal tapped into the Talgua River.

Because the flow in the irrigation canal is not sufficient for full power generation during the entire year, a 100% capacity backup diesel generator set was integrated into the overall electric system. This diesel generator can be paralleled with the hydroelectric power plant to supplement its output if necessary. Also, during times of extreme low water, maintenance of the water supply system, or hydroplant shut down,

[1]Mr. Campbell is the Project Engineer, a Consulting Civil Engineer from Conroe, Texas
[2]Mr. Wright is the Project Electrical Engineer, Radio Station HCJB, Quito Ecuador.
[3]Mr. McKay is the Overall Project Consulting Engineer, Duke Power Company, Charlotte, North Carolina.



SITE PLAN

FIGURE 1

the diesel generator can supply the needed power to keep the vocational training shops running and the refrigeration equipment functional.

Introduction

This project is a case study in the re-development of a small isolated hydroelectric power system for an educational and vocational training school and farm using an existing irrigation canal as the source of water, and an available head of approximately 14 meters. This particular project was necessitated due to the impending failure of an original 50 KVA James Leffel Samson 17 turbine/generator installed in 1970. This machine after 20 years was badly worn and was producing less than half the capacity needed for the operation of the school, farm and shops.

Project Description

The project itself is not just a hydroelectric power re-development. The entire project involved widening, and extending the existing irrigation canal, constructing additional facilities for gravity irrigation of an additional 40 hectares of land, constructing a reinforced concrete drop structure, and a 4 hectare storage reservoir having compacted earth embankments. Water from the reservoir is used for hydroelectric power production, and fire protection. The irrigation water is diverted at the end of the 3.2 km canal just before the flow enters the storage reservoir. Once the water enters the reservoir it is held and used for daily electrical production needs on a 24 hour basis. The electrical power is transmitted overhead on the school's own distribution lines. The electrical power is produced by an Ossberger Crossflow Turbine/Generator unit located in a separate powerhouse operating at an average head of 12.0 meters. The electrical controls system is a hardwired control panel built in the U.S. by SECAS, Inc. A 125 KW diesel/generator unit was installed as a back-up emergency source of power if the hydro station is out of service. The diesel/generator unit can be paralled with the hydro if necessary.

Civil Design

The civil engineering, and design was completed with as few drawings as possible. The major decisions regarding changes from the design drawings were made and implemented in the field by the Project Engineer. The civil design was simple in concept, using simple structures to complete the necessary work. All civil works and structures including all mechanical systems were shown on a total of 13 drawings. From these 13 drawings the entire project was built. The project was finished within 12 months and was an unusual mix of state-of-the-art hydroelectric generation technology, with developing world culture skills, and schedules, blended with well-intended, sometimes skilled, 2 week volunteer teams and equipment operators from the U.S. The civil work

had to be divided into three major phases, to accommodate the 5 month dry, and the 7 month wet season cycles of this part of Honduras.

Phase 1 consisted of temporarily rerouting the last 915 meters of the original canal to allow the old hydro turbine unit to continue operation, while the canal was extended and construction began on a portion of the south containment dike for the new reservoir. The extended canal turned 90 degrees into a 5 meter drop structure which dropped the flow into the new and enlarged reservoir. The new reservoir is 6 meters deep and has a capacity of 215,900 cubic meters. Phase I began in January of 1990 and ended at the end of May with the advent of the rainy season.

Phase 2, started during the wet season, consisted of the construction of the turbine-generator powerhouse, and the rebuilding of the entire farm school compound electrical distribution system, consisting of approximately 823 meters of overhead high voltage line and 1067 meters of underground low voltage distribution service. The construction of the powerhouse was greatly simplified by making a field change in its location. The plans had indicated a typical installation, which would have put the building in the lowest, wettest area below the dike in the river flood plain. The construction would also have required crossing the existing discharge channel from the small existing powerhouse.

An accurate topographical survey with the new intake structure, penstock, and powerhouse plotted, indicated there was just enough room to relocate the structures thus solving the wet excavation problem. This location did require cutting into an escarpment with a 6 meter cut, but this soil was used to construct the access road to the powerhouse, and do some useful filling in other needy areas on the farm and around the compound. During construction, the site remained high and dry and well drained, even though ground water was encountered in the last three feet of powerhouse excavation.

The first powerhouse concrete pour was a 30 cm thick foundation floor for the draft tube chamber. With the existing ground water seepage, this was a "wet pour", partially under water. With the use of pumps, the following wall pours were made, removing the water just enough to check and remove any sand and silt from the form bottoms. Beginning with the first wall section pour, the same set of 4.2 meter by 2.4 meter plywood forms were used through the rest of the job. The forms were reused 14 times, and very roughly treated, having to yank them out of sloughed soil numerous times, with a Komatsu excavator. The powerhouse was finished in December, in time to install the new Ossberger turbine/generator unit. Mr. Charles Snyder, a volunteer from Canada, experienced in such work, installed this equipment in January, 1991.

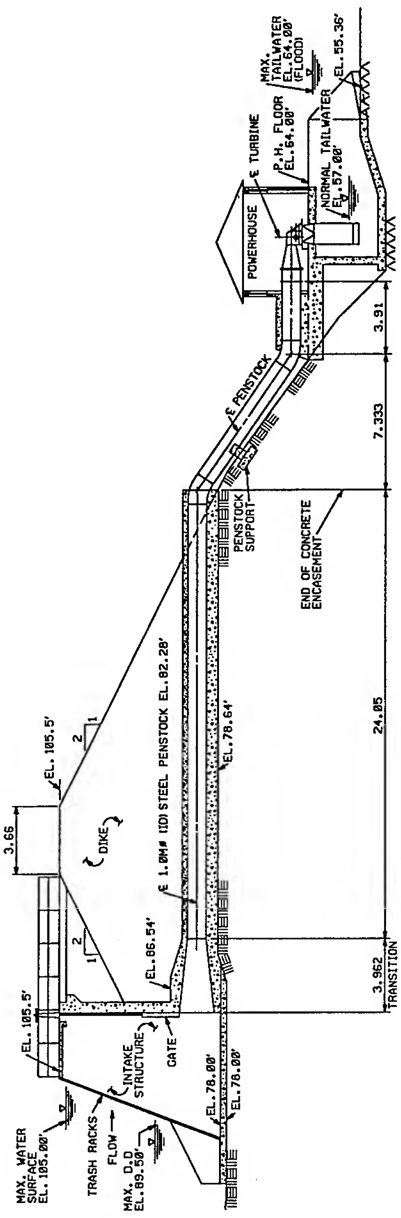
The third phase was the completion of the reservoir construction. It was performed in the dry season of January through May of 1991. During this time, most of the earth work for the reservoir dikes was put into place, and the concrete intake structure, steel penstock, and concrete emergency spillway were constructed. Figure 2 is a crosssection through the major structures.

Electrical/Mechanical

As stated earlier, a small antiqued hydro plant and old diesel generator set each rated at 60 KW generated the existing electrical power at El Sembrador. The machines could not be paralleled. Most of the protection and metering circuits for the machines were not functional. The worst of which was the regulating governor on the old hydro turbine. One of the students at the school manually controlled the opening of the wicket gates on the old turbine to control the speed of the machine. Above ground and buried cables comprised a low voltage distribution system operating at 208 VAC three phase. The capacity of the old generation equipment was not sufficient for the future expansion of the school.

Nothing in the original existing electrical system was re-used except the interior wiring of the buildings. Construction of a high voltage powerline with 18 pole transformers, and rain proof distribution panels connect the new hydroelectric plant and diesel generator to the new low voltage distribution system. A new service entrance which included the underground feeder and service entrance box with circuit breakers upgraded each building's electrical system. The scope of the project also included outside lighting to enhance the security of the entire facility. Since the school includes 32 buildings spread out over 40 hectares, the efficient distribution of electrical power required the high voltage power line. The routing of the high voltage power line is around the perimeter of the buildings away from the areas where the large farm equipment is stored and maintained. The main vocational training building 1.2 KM from the hydroelectric plant, contains many motor loads some with a high starting KVA. Having a "stiff" electrical supply near that building is essential for its productive operation.

Rainproof low voltage distribution panels mounted near the base of each pole which supports a pole transformer distributes the electrical power to the underground distribution system. This eliminates the need of electrical maintenance personnel to climb poles or make connections near the high voltage and transformers. Individual circuit breakers within the rainproof distribution panels protect the underground feeders to each building. This system reduces the possibility of a worker coming in contact with an underground feeder cable and being seriously injured. Moreover, each building can easily be isolated without shutting down the powerline or switching a pole transformer off. Overhead service drops



PROFILE THRU INTAKE, PENSTOCK, DIKE AND POWERHOUSE

FIGURE 2

posed a safety problem for the large farming equipment present at the school.

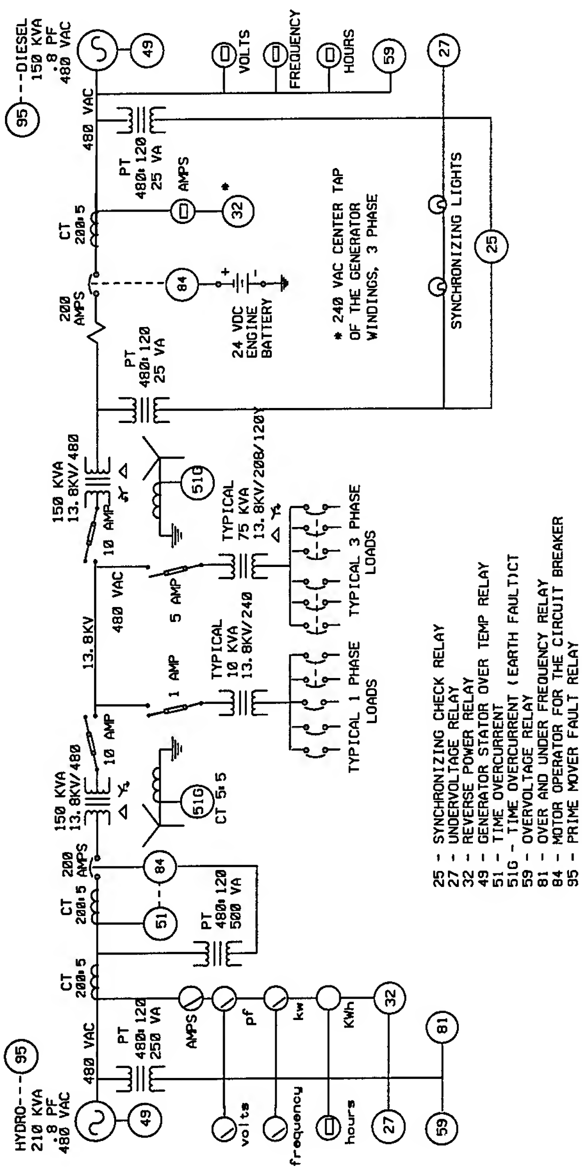
Continuous electrical power is desirable but the school can tolerate electrical power interruptions up to 30 minutes. The need for backup power to run the refrigeration (food storage for 200 Honduras boys and the staff) and energy to pump potable water was a concern. Therefore, the school management decided on a full electrical capacity diesel generator set for emergency power. A D3304TA Caterpillar unit rated at 125 KW electrical was purchased. Because of the river water being diverted for irrigation for rice fields at the farm, and low water flows during the dry season, the diesel generator set occasionally needs to be paralleled with the hydroplant. Hydroplant remote controls, manufactured by SECAS, Inc., located at the diesel generator are necessary to synchronize the machines and balance to the loads together.

The vocational training building has 75 KVA of normal electrical demand. Equipment ranging from machine shop lathes and welding machines to small electric hand tools in the wood working shop are used simultaneously. The turbine/generator supplier selected an oversized KVA generator to assist in the starting KVA requirements. In addition, the bigger frame size generator came with larger bearings to support the required flywheel.

The specifications of the generator are as follows:
Three Phase Synchronous generator, drip proof, tropicalized, 210 KVA @ .8 pf, WYE connected, Synchronous speed 1200 RPM, voltage 277/480 VAC, frequency 60 Hz, brushless AC exciter with six rotating diodes mounted on the generator shaft, solid state voltage regulator, manufacturer AVK of Germany. The generator control panel was built by SECAS, Inc., in North Carolina.

Figure 3 is a one line diagram of the electrical system.

The controls for the generator and powerline centers around the need to protect personnel and machinery in the unlikely event of an electrical fault. The powerline is WYE connected through a delta/WYE step up transformer having the neutral grounded on the high voltage side. Dedicated metering on the SECAS control panel monitors voltage, current, frequency, power factor, and KW. Along with under/over voltage, under/over frequency, and over current devices, ground fault current at the generator circuit breaker is also monitored. Also as part of the monitoring system, an annunciator package with battery back up memory provides the operator with an indication of a shutdown. 95% of the time the hydroelectric plant runs unattended.



125 KW HYDROELECTRIC T/G UNIT
SINGLE LINE DIAGRAM

FIGURE 3

A group of volunteer powerline workmen constructed the high voltage powerline. The materials for the powerline were purchased in USA and imported into Honduras under government permission. Using USA made parts facilitated the quick construction of the powerline and adherence to REA standards. At the time that the diesel generator set was commissioned the high voltage powerline was energized for the first time. The diesel generator set was the source of prime electrical power for the school while workmen completed the construction of the main hydro reservoir.

The specifications for the hydro turbine are as follows:
Turbine: Two cell (1/3 and 2/3) crossflow turbine, 204 HP, 273 RPM, Head 14M, bevel gearbox speed increaser 273 / 1200RPM, and a 614 kg flywheel. The turbine manufacturer was Ossberger in Weissenberg, Germany.

Ossberger supplied the frequency control governor and associated circuitry. The speed control governor utilizes an adjustable, electronic PID controller which gives flexibility in matching the governor response for picking up and shedding load. Conserving water at various load levels and being able to parallel with the diesel generator were the major reasons for selecting a frequency control governor.

Construction

The project is 210 kilometers from Tegucigalpa (4 hours by bus or truck). The main road crossed three mountain chains and always had several areas under repair without barricades, lights or warnings. The cement plant for Honduras and the refinery for fuels is on the north coast of Honduras, at San Pedro Sula and Cortez, making them 500 kilometers from the school by highway. The reinforcing steel was made in Tegucigalpa, in a recycling mill. It was rolled in millimeter thicknesses, not inches. Some of the rebar came from Nicaragua in a barter arrangement with the Honduran government trading Honduran dressed poultry for Nicaraguan steel. All the above is to indicate the distances and shipping difficulties that were encountered in the project.

All 382 cubic meters of concrete for the five major structures in the project, (New Irrigation Structure, Drop Structure, Intake Structure, Penstock encasement, Powerhouse) were batched by hand labor, shoveling river gravel into eight 19 liter plastic buckets, lifting them into a tractor drum mixer, adding 26.5 liters of water, followed by a bag of cement. This operation was performed an estimated 2500 times for the concrete work. Seven to nine cubic meters per day was good production. During the rainy season without earth work in process, the Komatsu excavator bucket was used to deposit the mixed concrete into the powerhouse slabs and wall forms. During the dry season, with the

Komatsu excavating soil full time, it was used only once for placing concrete, to top out the last meter of upper walls and the deck section of the intake structure. The lower 8 meters of intake structure walls were all poured with the tractor mixers backing up onto earth ramps which were temporarily added to the inside face of the earth dike section enclosing the structure.

The 111,000 cubic meters of earth work for the 1900 linear meters of reservoir dikes for the approach canal, and reservoir was excavated by the one Komatsu 220 Excavator. The material was obtained by digging out the interior of the reservoir area to gain both material and storage volume for power generation. The major earth haulers were two large dump trucks. The trucks were being repaired much of the time, since they were not intended for this type operation. Normally, the trucks would be loaded in the reservoir bottom and would circle up and down ramps to the spreading areas on the portion of dike being raised. A single D-5 dozer spread the dumped material in lifts which were compacted by successive passes of the loaded dump trucks. A farm tractor-pulled sheepsfoot compactor was used also. The loaded trucks provided 95 percent of the compactive effort. Fortunately, most of the excavated material was naturally graded as a sandy clay or a clayey sand and compacted well with it's natural moisture content.

Approximately 50 percent of the time, the Komatsu Excavator was operated by volunteers from a Georgia work team. Eighty percent of the time the two trucks were operated by volunteers from Georgia, Texas, the Carolinas, Ohio, Indiana or Pennsylvania. The farm tractors used to pull compactors, scrapers, levelors, water tank wagons, and even when operated as concrete mixers and haulers were operated 75 percent by volunteers. During the hundreds of days and thousands of manhours more than 160 volunteers, many of them, total strangers, worked together operating trucks, tractors, dozers and placing concrete on heights, in wet slippery holes below grade, on the steep slopes, and in tight crowded and dangerous places. During all of these operations, there was not one serious accident or lost time injury.

Conclusion

The full cost of the 125 KW project was $860,000. The alternative was to install a similar capacity D/G unit and operate it on a basis similar to the hydro. A simple economic analysis of these two systems clearly showed that the 125 KW hydroelectric installation would pay for it'self within 10 years of operation, as compared to operating a similar capacity D/G system for 10 years.

Computer Aided Design (CAD)
for
Small Hydroelectric Projects

Norman A. Bishop[1]

Abstract

This paper outlines the specific use of Computer Aided Design for Small Hydroelectric Projects and its benefits in terms of improved productivity and design quality. Computer hardware and software requirements, and drafting and design quality control requirements, are discussed. A case study is provided which documents the additional improved productivity when the design drawings are available from a similar generating station (Small Hydroelectric Project CAD Library) as the basis for the design of a new generating station. These benefits have been documented for retrofit, redevelopment and new hydroelectric stations.

Introduction

During the 1950's and 1960's, an Engineer prepared a pencil sketch, a drafter prepared a pencil drawing and later the pencil drawing was transferred by ink tracing to linen. The inked linen provided a durable drawing with a relatively long life. Unfortunately, any hydroelectric engineer who has worked on older linen drawings has experienced handling damage. To modify an older drawing can also be difficult and frequently you solve the drafting difficulty by preparing a new drawing on a wash-off mylar rather than working on the linen original. The manual design techniques of yesterday added quality to the design by tedious dedicated workmanship. The labour hours and materials required to prepare a design drawing were significant and many of the design improvements experienced in recent years are a direct result of available technology and economic application of technology.

[1]Manager of Engineering, Stone & Webster Canada Ltd. and KST Hydroelectric Engineers

The concept of drafting and design productivity became more important as design labour cost rose during the 1970's and 1980's. T-squares were replaced by parallel bars, varitypers replaced manual lettering, and pin-bar overlay drafting replaced individually drafted design drawings. At the same time, reproduction techniques were changing and improving. The use of wash-off mylars and stick on details are examples of improvement. The use of these techniques resulted in reduced drafting and design time, and reduced the cost of design production. The slide rule was replaced by hand held calculators, and calculators became programmable calculators. The greatest change has come as the result of the Personal Computer and its evolution. The most significant design productivity gains and record management improvements resulted from the use of CAD in the preparation of hydroelectric designs. CAD allows professionals to use multiple 2-D views, 3-D views, intelligent drawings and the ability for Computer Aided Engineering (CAE) software to be linked to design software. The CAD design drawing resulting is a hard copy original and a record electronic file.

The quest to reduce design costs and improve design quality by productive improvement is a concept familiar to every manager and design professional. CAD can improve design quality and productivity. If the past is any indication, constant future improvements can be expected.

Getting Started

The computer hardware and software technology has been advancing significantly and today ready-to-use or semi-custom CAD systems are available for purchase from specialty hardware suppliers. Over the last 10 years, the CAD hardware and software industry has been able to provide design professionals with practical and economically usable CAD packages. Today, specifying the CAD hardware and software is relatively easy once the design requirements are known. Finding experienced CAD professionals is frequently a more difficult problem. Most CAD users have two to three years experience, and are younger and less experienced personnel. Unfortunately, the more experienced design personnel are frequently not CAD literate. Prior to hiring or training CAD personnel and acquiring computer hardware and software, it is necessary to prepare a detailed assessment of needs. Under this assessment, the work requirements are matched with personnel, hardware and software. Human resource requirements are assessed. Existing staff training is outlined and the requirements are identified for staff acquisition.

When introducing any new technology, the manager needs to recognize that careful planning is required so that the introduction and implementation of the technology can be done with a minimum of adverse impacts. To successfully implement CAD and obtain quality designs, design personnel are required, with both CAD knowledge and design experience. Two years ago, we began a Joint Venture Company specializing in small hydroelectric development in Ontario,

Canada. The parent companies of this joint venture were each experienced CAD users and executive management strongly supported CAD and Computer Aided Engineering (CAE). The parent companies have main frame, workstation and network CAD and CAE systems and have extensive CAD libraries. It was decided to build on the parent companies' strength, yet customize the staff and CAD planning to the special needs of small hydroelectric work.

A CAD Business Plan process was conducted as shown on Figure 1, the CAD Assessment and Business Plan. The assessment of needs indicated that PC workstations provided the most flexible, easily implementable, and cost effective solution. CAD or CAE could be performed on the workstations which offered work planning flexibility. This approach also allowed for compatible communication with each parent company and the clients via modem. Also, network communication was established for printing and back-up filing.

The PC workstations were purchased and fully operational within one month from order. The basic PC workstation hardware specification is provided on Figure 2. This approach provided rapid flexibility for growth in design workload. As additional work is acquired, additional workstations are added to the network. The PC workstations are relatively portable and can be relocated and operational within an hour. Figure 3 shows the Joint Venture PC CAD workstations network. The PC workstations are connected to an ethernet which provides access to printing and plotting devices, and computer file back-up devices. Network wiring is 10 Base T with network access available in each engineer and designer work area. Figure 4 is a typical PC workstation photograph. To meet short term needs, parent company CAD compatibility allows workload peaks to be met with parent company CAD personnel and equipment, and the flexibility to defer computer hardware and software acquisition and CAD personnel requisitions until long-term CAD requirements are known. In this manner, personnel, hardware and software acquisition could be planned and implemented to assure workforce stability, high utilization and value. Intensive planning and design needs during plant outages can be met by this approach, allowing designs to be prepared and revisions made on the shortest possible schedules.

The needs assessment also indicated that there were more available trained CAD personnel in Ontario with a working knowledge of the AutoCAD software. AutoCAD was reviewed to determine if it would meet the requirements for small hydroelectric design work and it was found acceptable. Ontario surveyors, engineering and architectural sub-contractors, and equipment suppliers were found to also be AutoCAD users or had compatible CAD software. All software suppliers are constantly improving their products and finding CAD software which is compatible with sub-contractors is an important advantage. CAD compatibility is an important design and construction team work factor. Imagine if each engineer spoke a different language and the communication problems that would

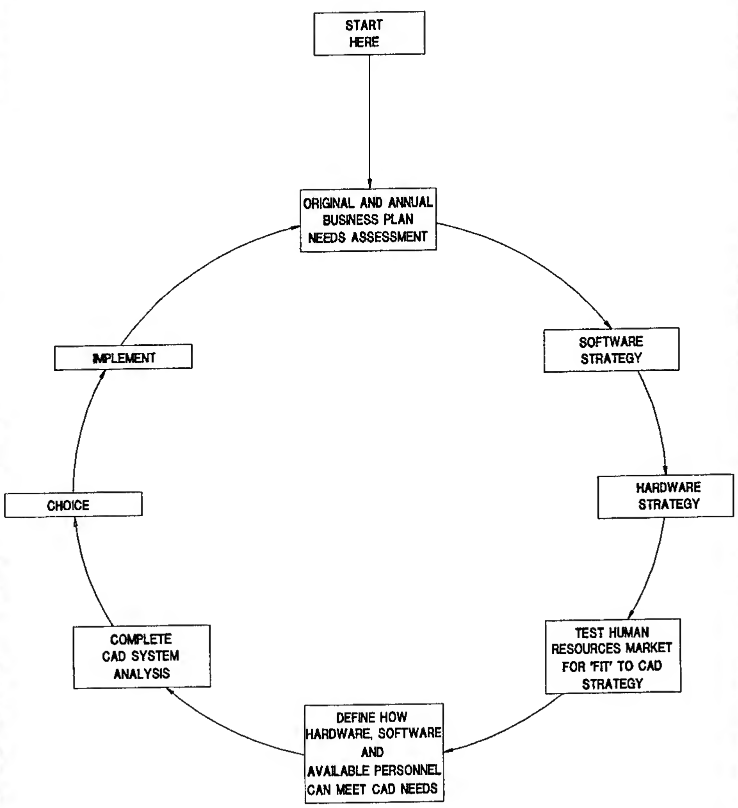


FIGURE 1 - THE CAD ASSESSMENT AND BUSINESS PLAN

CAD FILE : KST-CABP.DWG

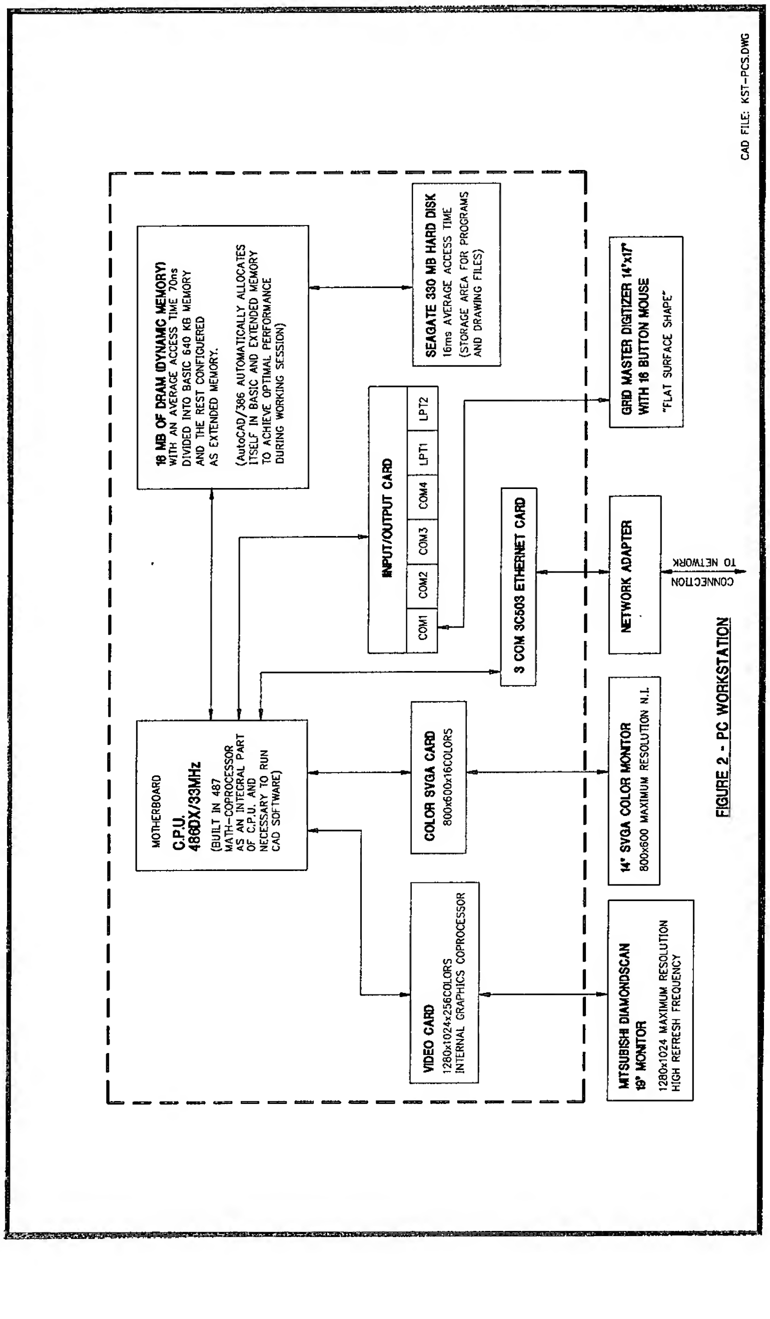


FIGURE 2 - PC WORKSTATION

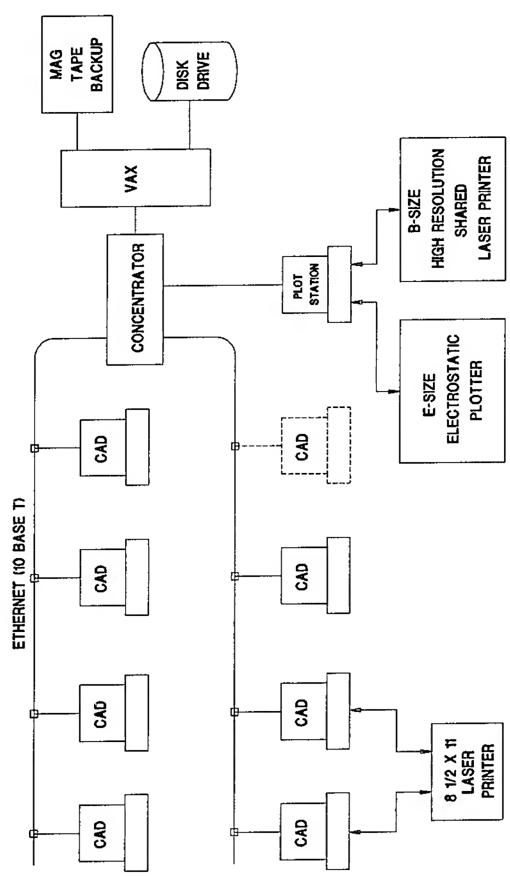


FIGURE 3 - CAD PC WORKSTATIONS NETWORK

CAD FILE: KST-NTWK.DWG

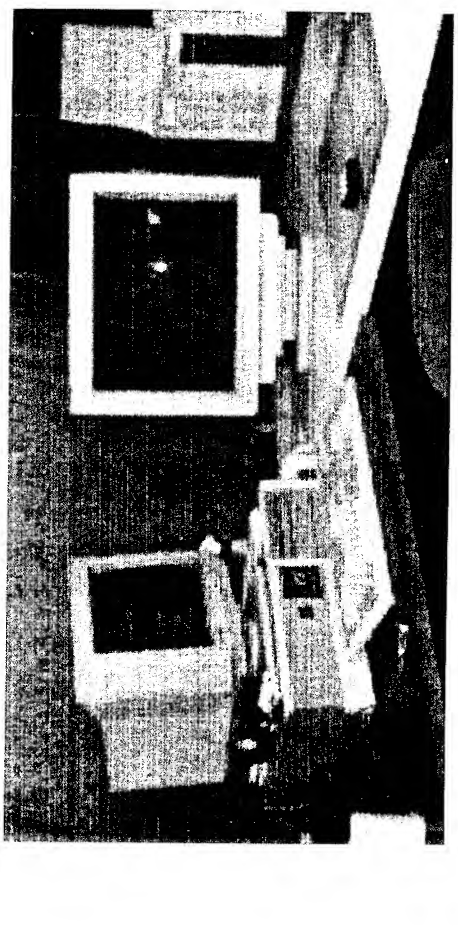


Figure 4
Typical CAD PC Workstation

result. Hardware and software compatibility is a key success factor promoting quality by avoiding communication problems and duplication of effort.

Building the CAD Teams

The success of CAD implementation and continued improvement is a result of teamwork. The CAD team is comprised of management and design professionals including engineers and CAD designers. Each member of the CAD team provides an important function in the design process.

In a CAD design organization of 25 to 75 professionals, a decentralized management structure offers distinct advantages of low management overhead and allows the CAD design professionals to exert control over their portion of the CAD resources, and use these tools to meet the requirements of today and tomorrow. Drafting and design standards, and design procedures are established and monitored. The objective is to establish a constancy of team purpose in which innovation, training, continuous improvement, and state-of-the-art CAD computer hardware and software are maintained. Quality is monitored in terms of production schedules and budgets, value engineering, productivity and construction costs. In small hydroelectric work, simple, cost effective concepts which will require no or low long term maintenance are stressed. Adjustments are made throughout the design process to promote continuous improvement.

Under the decentralized management structure, management is an important function. Management is charged with the establishment of the design and drafting standards and procedures. Management directs, coaches and supports the CAD design professionals with resource functional planning, work scheduling and budgeting. Encouragement is given by offering on-the-job training and tuition support for continued education. Management fosters teamwork, constancy of purpose, quality and continued improvement.

The management and design professionals must be committed to CAD. In the Joint Venture Office, no manual drafting is allowed during design drawing production. Engineering calculations and sketches are encouraged to be computerized. Engineers and designers are encouraged to be CAD trained. CAD software is available on the engineering personal computers as well as engineering software on the design PC workstations. CAD literate engineers are preferred when considering candidates for available professional positions. On-the-job training and inter-team project training encourage teamwork.

Each individual in a team contributes in a valuable and unique manner. Some will utilize the CAD facilities more than others. Management commitment to obtain high utilization of CAD facilities is an essential success factor. After two years with this commitment, 25% of senior engineers and every engineer at

an intermediate or junior level have become CAD literate. Our goal is to improve CAD literacy to 50% of senior engineers by 1994.

Monthly design and weekly project team meetings are encouraged to discuss problems and develop solutions. CAD requirements and training are identified and implemented. Engineers and CAD designers work together to identify and carry out the design work in a methodical and logical manner.

CAD operators have been traditionally personnel with a special talent to drive the CAD equipment. We have no CAD operators, only CAD designers. There is a distinct difference. CAD designers are professional designers and drafters who are also CAD operators. The CAD designers become better operators because of their ability to do creative design work directly on CAD. When a designer is working with a CAD operator, there is an arms length relationship which extends the communication path. Hands-on CAD work is the objective. As engineers become CAD literate, a similar experience was noted in terms of improved communication, productivity and quality.

Value engineering is also easier to perform when the engineers are CAD literate. An engineer can do value studies, independent of the design production work. During value studies, no changes to the actual production drawings are made until the value for the change is identified and the change is accepted by the design team. As an example of CAD design team effectiveness, a draft tube design was recently identified for a value investigation to possibly reduce concrete and rock excavation. The alternative design was identified and fully implemented in five working days including calculations, checking, review and turbine manufacturer's concurrence. The saving was about $50,000. In small hydroelectric work when budgets are tight, this capability is very useful.

Design Process

The design phases vary little from engineering discipline to engineering discipline. Some discipline design work tends to be more drafting intensive. The phases of drawing preparation are shown on Figure 5 and consist of Conceptual or Feasibility Engineering, Preliminary Design and Design Criteria, Design, Design Checking, Review and Approval. Once approved, the designs are released for bids and as a result of bidder questions and feedback, design revisions may be required. Once bids are received, the designs are issued for construction.
During construction, differing field conditions or other circumstances may occur which require construction revisions. Finally, as the construction and commissioning are completed, 'as-built' design revisions are made.

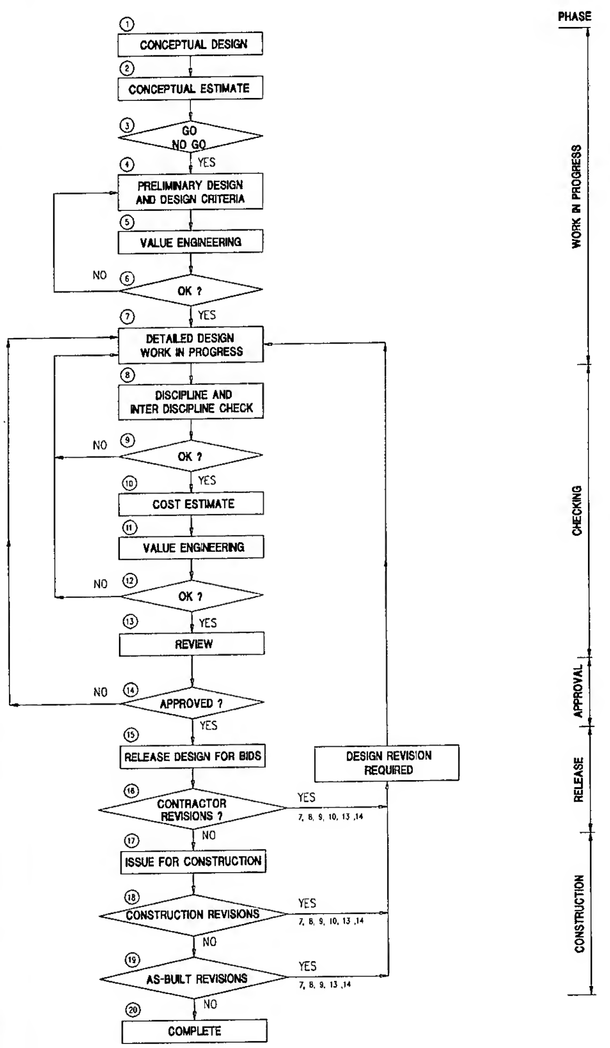


FIGURE 5 - TYPICAL FLOW OF DESIGN WORK

CAD FILE: KST-DIAG.DWG

Conceptual or Feasibility Engineering identifies the hydroelectric resource and defines the types of structures and equipment that will be required. To define the concept, hydrologic studies, power studies, geologic and soils studies, preliminary surveys, cost/benefit estimates and economic analysis, and other work are performed. Preliminary design includes project description, schedules, budget estimate, station capacity, number and capacity of units, special environmental requirements, design criteria, drafting requirements such as drawing sizes, letter sizes and other basic requirements. Also included would be detailed hydrology, detailed geology, surveys, supplier information and conceptual data from other engineering disciplines. Value engineering studies are performed. It is recognized that this information may change as specific designs are developed and refined, or as value engineering is performed. The detailed design process includes the preparation of the actual detailed calculations and design details. The design is then checked, drafting is performed and checked. The designs are reviewed and further value engineering is performed. The value engineering process may require further design, drafting and checking. Finally, the design is approved. Design revisions are also completed in a similar manner.

The term "productivity improvement" describes the improvement in terms of labour and cost savings of the entire engineering process from conceptual engineering through design approval and construction. Functional strategies are important to overall productivity improvement. An example of functional strategy would be to have survey information in a computer file which can be input directly into a CAD system or to obtain supplier information in CAD compatible files. Another example is the use of previously prepared details available in a CAD library. The CAD library is discussed in more detail in the following section. Functional strategies avoid work duplication and improve quality.

The turbine manufacturer may have an existing design or has already prepared a scaled outline of the proposed water passage and equipment. As a part of the equipment bid documents, specifications require the submittal of drawings in CAD files. CAD allows the use of a computer file of this information for the preparation of civil, mechanical or electrical drawings. This technique avoids drafting duplication.

Another approach to productivity improvement is the use of engineering software (CAE) which is compatible with the CAD software. Combining CAE and CAD allows the geometry of a 2-D or 3-D finite element analysis to be used in the CAD file. An example is the development of river channel transects in HEC-2 and use of these transects to define channel cross sections in CAD. This approach saves time in defining the geometry at bridges, structures extending into the river, headwater approach channels and tailrace channels. However, refinement of the CAD design files are usually required to obtain a quality design.

CAD Library

The use of a CAD library is an important functional strategy for productivity improvement. The CAD library is an organized archive of CAD files of drawings and CAD details. When one evaluates the elements of a design drawing, many items are repeated from design drawing to design drawing. Once these repeated elements or bits of information are available in a CAD file, they can be retrieved from the CAD library and utilized over and over again. Traditionally, these details have been redrawn or stick-ons used.

Another functional strategy is the use of standard details. Design standard details are details which are common to many designs which are used over and over again. These are details which are proven to be cost effective, buildable and of high quality. When standard details are used, it avoids design errors and improves design value. Standard details are a good start when a custom detail is required. CAD allows the designer to utilize a portion of the detail. Any portion of a detail which is used represents productive improvement when compared to beginning a new detail. This avoids duplication and allows the designer to pursue other designs. Also, this strategy can minimize mistakes.

A powerful and cost effective functional strategy is the use of similar or identical CAD designs for retrofit, redevelopment or new project work. For example, if a CAD library contains a variety of powerhouse or dam CAD designs which closely match a new project's design requirements, an Owner can save as much as 50% in design costs and have a construction-proven design. Of course, customizing a design to suit the uniqueness of a new project requirement is also important. This has been demonstrated in recent new and redevelopment small hydroelectric station design work. There are also design savings in this strategy for hydroelectric stations that require retrofit; however, the designs must address the existing station layout, geometry and other spatial conditions. The design savings are significant but usually less because of the additional customizing.

An example of design cost savings is the recent design of retrofit of three existing small hydroelectric stations which contained vertical open flume Francis units. Thirteen identical replacement inclined axis, bevel gear double regulated Kaplan turbines were purchased to replace four units for two stations and five units for the third station. The CAD design details of the initial station will be used for the other stations. Unique details include the intake, draft tube and cofferdams. The cost savings in design, equipment and construction are significant with this approach. Another example is the use of an S-turbine powerhouse redevelopment where an existing design was customized to suit actual requirements. This strategy has been used in penstock, spillway and dam rehabilitation and powerhouse control automation work. CAD techniques have been demonstrated over and over to be successfully productive and cost effective.

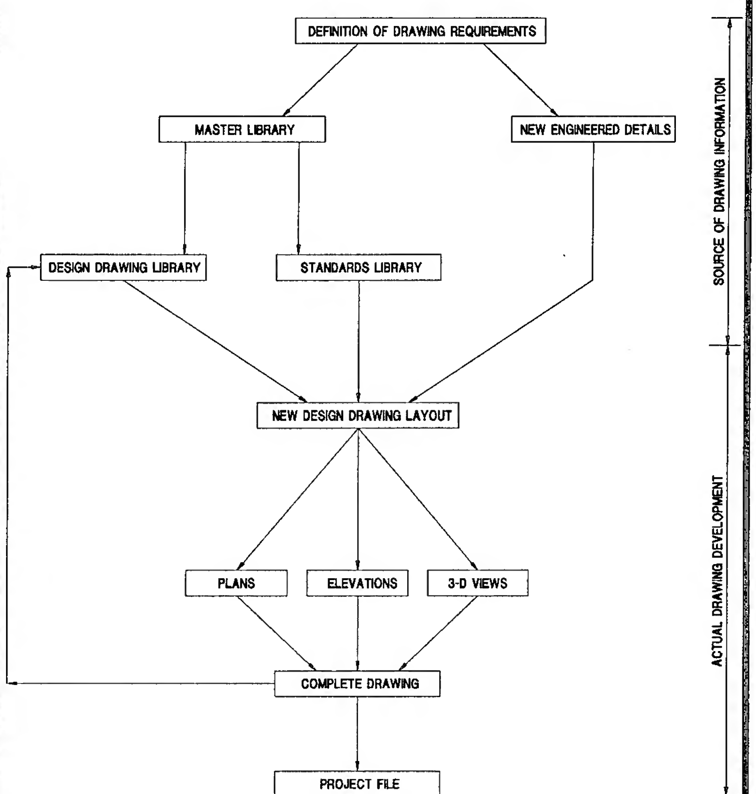


FIGURE 6 - USE OF CAD LIBRARY IN DESIGN PROCESS

CAD FILE : KST-ACLB.DWG

Today, a good small hydroelectric CAD library would contain powerhouse designs for different turbine types including bulb, S-style, pit, Francis, Kaplan, Propeller and Pelton. Included should be both horizontal and vertical designs. Generator arrangements should include high speed with speed increasers and direct coupled styles. A variety of dam and spillway types should be included in the CAD library. A spillway CAD library would include various gated and ungated arrangements, with details of vertical and radial gates, hoists and other devices. This approach is applicable to substations, switchyards, transmission and distribution lines. The rule is never start a new design if you can begin a design from an existing CAD library file. New engineered details are the most expensive designs. If you can begin from an existing design that is proven by construction and operation, quality of design and construction value is proven. This approach offers better design and construction quality and productivity. Experience teaches us that mistakes and impractical or unbuildable solutions are more frequently made when beginning designs from new.

CAD Digitizer or Data Tablet

There are many input devices which can be used to draw points, lines and symbols on the screen including the keyboard. One of the most productive input devices is the CAD digitizer or data tablet. This device has an electromagnetic or other grid type that can sense the position of a stylus or smart mouse which controls the cursor on the computer monitor. The digitizer is a standard device for our CAD PC workstations. The use of a digitizer is another functional strategy. Experience indicates that some additional drafting work is usually required after an existing ink or pencil drawing is digitized to complete line work and touch-up details.

Scanning

Another tool is the use of a scanner. This device is used to scan a picture, drawing or document to obtain an electronic record file. When the original is scanned, the electronic image is in a RASTER format or a series of unconnected dots or pixels. A line is a series of adjacent pixels with the same colour. However, in a raster format, a line is not identified for the computer as a line and an automatic vectorizing program is used to identify the adjacent pixels as a line. This explains the difference between a "RASTER" and a "VECTOR" image. For example, an AutoCAD file which describes a line is a vector file. The vector file permits you to assign or calculate line length and areas of closed polygons. The picture of a typical CAD PC workstation shown in Figure 4 was printed from a "RASTER" format file. The other figures are "VECTOR" format files. The use of scanning and "RASTER" format files can be employed to architecturally depict how existing structures will look once modifications are made to the landscape or to the building exterior.

Component Menus

Drawing elements, components or details can also be requested from the computer hard disk file or network storage by keying a unique name. These elements can be combined within a menu which is displayed on screen. The CAD menu usually contains names which define icons or symbols which can be accessed by the designer by the keyboard or by a mouse. The menu is a mini CAD library which can be amended, copied, or used for CAD design work. The menu is customized to civil, structural, mechanical/piping and electrical/control design work. To assure consistency of our CAD designs, a custom menu has been developed where the menu is arranged in a logical, consistent and predictable manner. This menu has been customized to hydroelectric work.

Design Layers

A powerful design tool is layering. CAD files are prepared in actual size and then downscaled or upscaled for printing purposes. On the other hand, manual drafting is done to a scale and the scale varies from detail to detail. By drawing to actual size, CAD allows the designer to build a design drawing in layers. A layered drawing may include a layer defining contours, excavation, foundations, construction phases of structural work, columns, beams, mechanical and electrical, different equipment furnished by different manufacturers and other variations. A design layer is established as the designer inputs a component into the CAD design model. Layering is allowed to be specific to project types, however, procedures require that certain basic layers be established by engineering disciplines and equipment manufacturers. For example, by keeping the contour information in a layer, it allows you to make changes on the building footprint without modifying the contour's information. The use of design layers can also help the owner, designer and constructor visualize the sequence of the construction work for planning purposes. There are many uses and advantages of layering in checking and interference verification in congested design areas.

Layering also allows the use of a "RASTER" format file as one layer and the superposition of other layers to depict the physical design aspects. This is an effective tool for the designer when laying out project features like canals, penstocks, access roads, parking lots, transmission lines and switchyards, over aerial photos.

Real Results

CAD has been demonstrated to provide real productive improvement in small hydroelectric designs. CAD has been used on a variety of small hydroelectric and dam assignments with satisfactory results. Over the last two years, CAD has been used on more than 20 small hydroelectric projects.

Each project design has consistently high design productivity and design quality. Drawing production is typically 50 or less drafting hours per drawing, depending somewhat on the uniqueness of the design or spatial geometry. The actual trend of more recent design work is further productive drafting hours reduction with a 1994 goal to average 40 or less drafting hours per design drawing.

The CAD functional strategies depicted are real and implementable in small hydroelectric design work. Value improvement of CAD designs is a necessary part of the quality commitment. The CAD design project teams identify new hardware, software and functional methods for productivity improvement. On-the-job training and inter-project training will continue as part of the overall quality program. Monthly, and on a more rigorous annual basis, a team review is necessary of the CAD utilization, productivity and quality. Continuous adjustments are necessary when trying to implement any new technology.

The Future

Under current study is the application of new CAE software and the use of Geographical Information Systems (GIS) for small hydroelectric project work. Less than one year ago, an Owner Alliance agreement was initiated to furnish and install the water-to-wire equipment for some of the Ontario small hydroelectric work. This has offered an opportunity to openly discuss the traditional Consulting Engineer CAD design role and the Water-to-Wire Contractor CAD design/ manufacturing role and consider the elimination of further duplication of CAD/CAE efforts. Some co-operative CAD design work has been completed and additional Owner/Engineer/Manufacturer team design is a objective for future small hydroelectric assignments.

The expanded use of Raster and Vector format file overlays offers a number of future opportunities. The Raster and Vector layers can be used for a Digital Image Map (DIM) or a Digital Ortho Image (DOI). As-builts can be created in this manner, if existing features have not been previously as-built. The checker's design comments can be placed in a Raster file and layered into the design drawing vector layers. This is an especially powerful tool when many discipline engineers or multiple design offices are involved in a design. It also offers management a compact record of the checking and approval process which

is a possible alternative to the conventionally used stick and record files of check prints. Productive use of office space is another functional strategy.

Conclusion

CAD is useful and productive in small hydroelectric design work. Through planning and teamwork for retrofit, redevelopment and new small hydroelectric work, CAD design is cost effective with quality results. CAD functional strategies for productivity and quality improvement have been presented which can be implemented by the small hydroelectric project designers. The use of Computer Aided Design is effective in reducing design duplication, and aids the inter-team planning and communication of ideas between an Owner, Consulting Engineer/ Construction Manager, Equipment Suppliers and Contractors.

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VICTORIA DAM REHABILITATION

By Robert D. Reynolds[1], M. ASCE; Robert A. Joyet[2], M. ASCE; and Max O. Curtis[3]

Abstract

Victoria Dam, constructed in 1930, consists of a 37 m (120 ft) high, concrete multiple arch-buttress dam. Over the years the non-air entrained concrete experienced significant freeze-thaw deterioration and a decision was made in 1989 to replace the dam. Construction of a new roller compacted concrete (RCC) dam immediately downstream was found to be the best economical and technical approach. The new gravity structure could be tied into the existing gated spillway and penstock intake structure on either abutment, saving the additional expense of replacing these structures. This paper discusses the design, construction, and performance of the RCC replacement dam.

RCC Design

Victoria Dam is located near the village of Rockland in the Upper Peninsula of Michigan. The project is owned and operated by the Upper Peninsula Power Co. The dam is a straight axis RCC gravity structure, having a crest length of 100 m (330 ft), including a 40.4 m (132.5 ft) long ungated overflow spillway. A typical cross section is shown in Figure 1. The 36,000 m^3 (47,000 cy) dam is constructed of RCC

[1] Project Engineer, Stone & Webster Engineering Corp., Denver, CO.
[2] Principal Geotech Engineer, Stone & Webster Engineering Corp., Denver, CO.
[3] Superintendent of Production, Upper Peninsula Power Co., Houghton, MI.

encapsulated with a facing of conventional, air entrained concrete to improve surface durability and aesthetics.

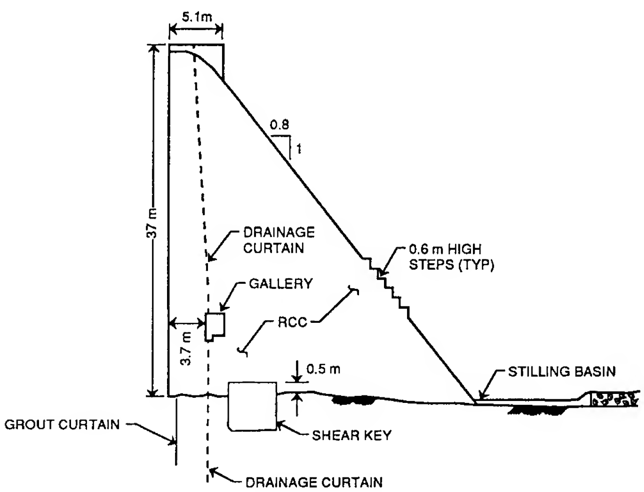


Figure 1. Typical Dam Section

The RCC was designed to have 90 day and 1 year unconfined compressive strengths of 10.3 MPa (1500 psi) and 13.8 MPa (2000 psi), respectively. The required lift bond strength cohesion with an assumed friction angle of 45 degrees was 0.34 MPa (50 psi). This was considered readily achievable for an RCC with a compressive strength of 10.3 MPa (1500 psi), therefore, laboratory design shear tests were not considered necessary.

RCC mix design studies were conducted using 1.16 kN/m^3 (200 lb/cy) and 1.45 kN/m^3 (250 lb/cy) cementitious contents with 30, 40, and 50% flyash replacement. ASTM C150, Type II, low alkali, low heat cement was used in combination with ASTM C618, Class C flyash, which was available near the site. Concrete proportioning (sufficient cement paste available to fill voids between aggregate) was used. A summary of the compressive strengths is shown on Figure 2. A design mix of 1.31 kN/m^3 (225 lb/cy) cementitious content with 50% flyash replacement was chosen for construction.

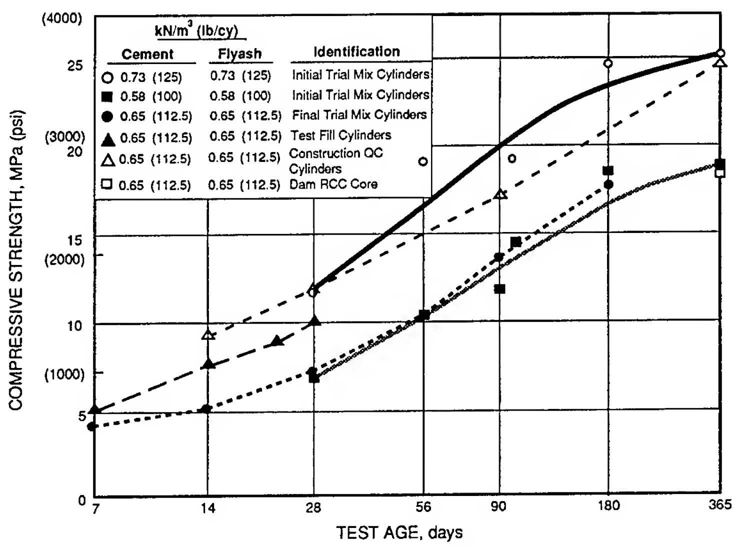


Figure 2. RCC Unconfined Compressive Strength Test Results

The use of Class C flyash, rather than Class F, raised some concern about the possibility of a faster RCC set and less working time. Mortar set tests were made to evaluate the benefit of using a water reducer/set retarder admixture (Conchem PDA-50). The penetration resistance test (ASTM C403) was used to evaluate initial and final mortar set time. Dosage rates of 118, 177, and 237 ml (4, 6, and 8 oz) per hundred weight of cementitious material were used. Results indicated that using 237 ml (8 oz) of PDA-50 increased the initial set time from 6 hours to 11 hours, and the final set time from 9 hours to 13 hours. For construction, 237 ml (8 oz) of PDA-50 per hundred weight of cementitious material was specified. Bid prices in March 1991 for this varied from about \$0.55 to \$1.15 per cubic meter (\$0.42 to \$0.88 per cubic yard) of RCC.

RCC aggregate was a hard, dense basalt (specific gravity = 2.86) quarried about 5 km (3 mi) away. A 51 mm (2 in) maximum size aggregate (MSA) was specified to reduce segregation. The RCC aggregate had about 35% passing the No. 4 sieve and 5% passing the No. 200 sieve. The RCC design mix was wet screened through a 38 mm (1.5 in) sieve for testing. Cylinders were cast in three lifts using either a vibratory table with an 89 N (20 lb) surcharge, or a hand-held Hilti TP-400 vibrating hammer having a total weight of 89 N (20 lb). The Vebe time for consistency was measured by a vibrating table using a 254 mm (10 in) diameter, 0.014 m^3 (0.5 cf) unit weight bucket with

a 222 N (50 lb) surcharge, and a 129 N (29 lb) RCC sample. The Vebe times generally fell in the range of 15 to 20 seconds using 1.16 kN/m^3 (200 lb/cy) of water in the mix. The Hilti hammer brought paste up around the plate in 6 to 8 seconds. The hammer was much easier to use, resulted in approximately the same RCC densities and unconfined compressive strengths as the vibrating table, and was therefore selected to prepare cylinders during construction.

A decision was made early-on in design to use contraction joints to control transverse thermal cracking through the dam. A simple thermal analysis was performed based on worst case assumptions. Worst case conditions used for the analysis were:

• maximum RCC mass heat gain	20°C (35°F)
• maximum foundation restraint	R = 1
• maximum allowable RCC placement temperature	24°C (75°F)
• minimum RCC equilibrium temperature	7°C (44°F)
• minimum RCC allowable strain capacity	100 millionths

These extreme values were used to evaluate the overall tensile strain in the dam. The calculated overall tensile strain was then compared to minimum allowable strain. Potential crack distribution and size were then evaluated based on cumulative excess strain over the dam's length. The worst case analysis suggested that four cracks of about 4.75 mm (3/16 in) width would form.

Five contraction joints were installed. The joint locations were based on foundation topographic breaks, dam cross section variation, and a maximum joint spacing of 25 m (84 ft). Between the contraction joints, vertical crack inducing control joints were installed on the upstream face and caulked after completion of construction. The control joints were initially located on 4.9 m (16 ft) centers. Intermediate cracking of the facing concrete was noticed during the early stages of construction, so the spacing of the control joints was reduced to 2.5 m (8 ft). The joints were sealed with a 2-part, non-sag urethane. Details are shown on Figure 3.

The RCC was designed to be placed in 30 cm (12 in) compacted lifts. Bedding grout was placed between each lift in the upstream 5.1 m (17 ft) section to reduce seepage potential along the lift lines. A drainage system was installed to reduce hydrostatic pressures in the foundation and dam. The system includes a drainage gallery, foundation drains, and drains within the dam.

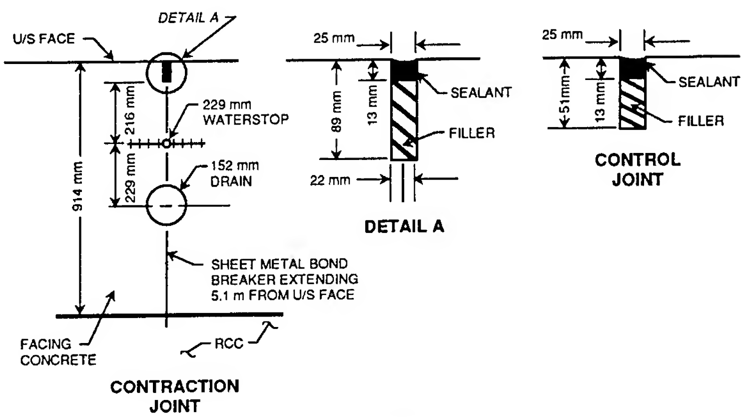


Figure 3. Plan-Contraction & Control Joints (NTS)

The foundation consists of sandstone with occasional thin clay seams dipping across the site at 15 degrees. These clay seams are the controlling factor in foundation stability. A shear key was installed to provide necessary resistance. The conventional concrete key extends a minimum of 2.1 m (7 ft) below the general rock surface at the downstream toe, is 4.6 m (15 ft) wide, and extends 0.5 m (1.5 ft) above the foundation so that it also keys into the RCC.

Construction

The 36,000 m^3 (47,000 cy) of RCC and facing concrete were placed in an approximate 8-week period in the fall of 1991. The contractor worked six days per week in two 10-hour shifts per day with a 2-hour break between shifts for maintenance. RCC batching used a 3.4 m^3 (4.5 cy) capacity Ross compulsory pug mill with triple bin weight batcher for aggregates, separate cement and flyash batchers, volumetric admixture batcher, a metered water line, and a separate enclosed operator's booth. Automatic batching was computer controlled with manual override capability. Adequate mixing was achieved in 30 seconds.

The RCC was transported about 137 m (450 ft) to the furthest placement location by a 610 mm (24 in) wide conveyor and two Port-O-Belt Swingers as manufactured by Rotec Industries. RCC was discharged immediately behind the leading edge of the fill where a

Cat D3 spread it into about 360 mm (14 in) loose lifts. Each lift was compacted with up to eight equipment passes of a 89 kN (10 ton) double drum vibratory roller. The next lift had to be placed within 10 hours or 500°F hours, whichever was less, to prevent a cold joint. A cold joint required cleaning and placement of a nominal 12 mm (0.5 in) of 27.6 MPa (4000 psi) bedding grout prior to the next lift of RCC.

The test fill indicated that the laboratory trial mix was too wet for placement and the RCC had to be reproportioned to a lower water content of 1.05 kN/m^3 (180 lb/cy). This resulted in Vebe consistency times exceeding 35 to 45 seconds. Therefore, Vebe time was no longer an effective control parameter for RCC consistency or workability, since Vebe times exceeding 30 seconds are difficult to distinguish and are not meaningful. Consistency was continuously evaluated on a visual and behavioral basis. It was determined in the test fill that the ideal moisture content mix had a classic "pumping" behavior during compaction, resulted in a nearly smooth/nearly paste-filled surface, and created 12 to 25 mm (0.5 to 1 in) depressions at the edge of the roller drum. The RCC also exhibited a certain sound and slumping behavior when dropped from the conveyor. An overly dry mix exhibited significant segregation when both dropped and spread. An overly wet mix had a distinctive "sloppy" sound when dropped, and stuck to the vibratory roller.

Contamination of the fill surface was caused by loosening of the compacted surface by construction equipment and foot traffic, and by rain. The tracked dozer was not allowed to travel directly on the RCC, and rubber tire equipment was discouraged from making sharp turns. In addition, some contamination occurred from drippings off the overhead conveyor. Any loose, disturbed RCC was removed and the surface blown clean. Limited brooming and water flushing were used as both tended to leave non-cemented particles in the surface voids. These then needed to be vacuumed up or blown off.

When rain was imminent, compaction was completed as close to the spreading face as possible and the compacted surface covered with plastic sheeting. A number of times it was necessary to stop RCC placement due to damaging rainfall. The formation of a creamy paste at the surface during compaction signaled a shutdown. Samples of this paste had high moisture contents and 28 day unconfined compressive strengths of only about 0.76 MPa (110 psi), confirming its dilution. Cleanup consisted of brooming, air/water jetting, vacuuming up ponded water and blowing off loose particles.

Facing concrete is nominally 610 mm (24 in) thick. It consisted of 19 mm (3/4 in) MSA, 27.6 MPa (4000 psi), air entrained conventional concrete with water reducer and retarding agents. Facing concrete was

placed following RCC spreading and surface compaction. Edge compaction of the RCC was a problem due to safety concerns about running the compactor too near the edge, and due to squeezing out of the RCC beyond tolerances, which then required hand trimming. Wetter RCC mixes were particularly a problem, as was RCC edge placement over the upstream lift bedding grout which tended to "lubricate" the placement surface. RCC-facing interface compaction included application of an immersion vibrator into and normal to the face of the RCC edge on approximately 460 mm (18 in) centers and at least one additional pass of RCC compaction with a walk-behind double drum vibratory compactor.

The downstream step form system consisted of 9.1 m (30 ft) long weighted H beam sections set on edge with 610 mm (2 ft) high forms bolted to the flange. The forms were set on plywood strips. Surface friction was relied upon to prevent lateral movement. This system necessitated significant crane capacity for the longer reaches. However, placement was done quickly and required minimal labor.

Quality control of the RCC during construction consisted of full time visual inspection, as well as cylinder, nuclear density, mixer performance, and optimum compaction density (OCD) tests. The day-to-day assurance that design strength would be met was by continuous visual control of the mix and nuclear density testing to verify that the correct mix was adequately compacted.

Density was checked by nuclear density gauges at a minimum of six locations per lift. A double probe gauge (CPN MD-S-24 Strata Gauge) was specified, with a single probe gauge (Troxler 3411B) as a backup. The double probe gauge enabled the measurement of density across a particular depth interval, rather than vertically to the surface as the single probe gauge does. The target density for acceptance was 97% of theoretical air-free density, 24.56 kN/m^3 (156.4 pcf), or 98% of OCD. The 1500 densities taken resulted in an average RCC density of 25.07 kN/m^3 (159.6 pcf) which agrees quite well with the average density for the RCC core samples as shown in Table 1.

<u>Performance</u>

The reservoir was filled during the first quarter of 1992. Seepage and RCC temperatures were monitored regularly during and after filling. In addition, three 152 mm (6 in) diameter borings were cored through the dam in September 1992, about 1 year after construction, to evaluate the RCC condition.

Total seepage into the galley increased to a maximum of 52 liters/second (220 gpm) as the reservoir filled. This included a 5.0 l/s (80 gpm) concentrated leak through a near surface fracture in the left abutment foundation rock. The foundation fracture was grouted about 6 months later at which time the total gallery seepage was reduced to about 4.7 l/s (75 gpm), of which 31% was from foundation rock abutment drains, 27% from contraction joint drains, 23% from internal RCC dam drains, 12% from foundation drains, and 7% from gallery perimeter seepage. Seepage out of the contraction joint drains is probably due to seepage through shrinkage cracks in the upstream facing concrete which flows down along the more pervious interface at the facing concrete and the RCC, and out the contraction joint drains. About 95% of the flow out of the internal RCC drains is from the drains near, or partially through, the abutment rock. The drains through the central portion of the dam are dry. Scattered damp spots are visible on the lower one fourth of the downstream face of the dam.

The gallery length within the RCC is 40 m (130 ft) and one contraction joint is located in this section, approximately in the middle. This joint has opened about 1 mm (0.04 in) around the entire gallery perimeter. In addition, one other crack has opened 0.8 mm (0.03 in) across the gallery near the toe of the right abutment slope.

Eleven thermocouples were installed during construction to monitor RCC temperatures. The seven thermocouples in the internal portion of the dam showed an average 20°C (35°F) increase in temperature. The maximum RCC temperature was about 38°C (100°F) and occurred about 6 weeks after the RCC lift was placed. The internal portion of the dam had cooled to 18°C (65°F) about 16 months after construction and was still cooling.

One set of RCC test cylinders was made on each lift of RCC during construction. Density and unconfined compression tests were made on the cylinders at 14, 28, 90 and 365 days of age. The 1-year date is shown on Table 1.

Three 152mm (6 in) diameter core borings were also made in the dam approximately one year after RCC construction. The core indicated the RCC was generally well consolidated with only occasional small void concentrations. Void concentrations were more prevalent near the bottom of lifts, but none were observed to be either intense or continuous. Generally, the bottom portion of lifts appeared to be equally consolidated as the top portion. Bedded lift lines were generally able to be recognized in the core. Unbedded lift lines were at times unable to be identified.

Property	RCC Cores		RCC Construction Test Cylinders	
	Range	Average	Range	Average
Density kN/m^3	24.5 to 25.5	25.0	24.3 to 25.6	25.0
(pcf)	(156.2 to 162.3)	(159.0)	(155.0 to 162.8)	(158.8)
Approx. 1-Year Comp. Strength				
MPa	13.6 to 24.5	18.5	14.0 to 34.8	24.8
(psi)	(1970 to 3560)	(2679)	(2030 to 5040)	(3587)

Table 1 - RCC Density and Compressive Strength

Density, unconfined compressive strength, and direct shear strength tests were done on selected samples of the core. The density and unconfined compressive tests were made on 30 samples selected at random. The test results are compared to those from the construction cylinders in Table 1. The measured RCC average density from the core is about the same as measured in the cylinders. However, the measured RCC average 1-year unconfined compressive strength from the core is only about 75% of the average strength measured in the construction cylinders. Since both sets of samples had the same density, appeared equally well consolidated, and had adequate curing conditions, it is speculated that the lower strength in the core could be due to microfracturing of the initial bond formation during the initial RCC set caused by vibration from the 89 kN (10 ton) compactor on overlying lifts.

Direct shear break-bond and sliding friction tests were made on the 1-year old RCC core. Intact RCC, bedded lift lines, and non-bedded lift lines as well as broken bedded and non-bedded lift lines, were tested. The best fit shear strength results for each type of test are presented in Table 2.

Condition Tested	Break-Bond		Sliding Friction	
	Cohesion MPa (psi)	Friction Angle	Cohesion MPa (psi)	Friction Angle
Intact RCC	1.93 (280)	64	0.26 (38)	47
Intact bedded lift	1.59 (231)	69	0.10 (14)	44
Intact non-bedded lift	1.19 (172)	62	0.17 (24)	48
Broken bedded lift	---	---	0.30 (44)	30
Broken non-bedded lift	---	---	0.07 (10)	36

Table 2 - RCC Core 1-Year Shear Strength

Lessons Learned

- The double probe density gauge is preferred over the single probe gauge to provide the necessary information on the degree of compaction at specific depths of interest, rather than a weighted average between the surface and probe depth as with the single probe gauge.

- The use of Vebe time to control RCC consistency and workability was not appropriate for this mix. Visual observations during placement and compaction provided good control.

- Preparation of test cylinders using the hand-held Hilti TP-400 vibrating hammer was much faster than a vibratory table and gave equivalent results.

- There is no need to conduct expensive and complex computer thermal analysis on all RCC dams. A simple calculation and judicious placement of contraction joints may be sufficient.

- The unconfined compressive strength of the RCC as measured in the post-construction cores was only 75% of the strength measured in the construction quality control cylinders. It is speculated that this may be caused by microfracturing of the RCC bond formation during initial RCC set by successive lift compaction vibration.

CASE HISTORY OF SHERMAN ISLAND HYDRO BUTTRESS DAM

Jacob S. Niziol, P.E.[1] and Edward M. Paolini[2]

ABSTRACT

A staged program for safeguarding a deteriorating multiple arch buttress dam by construction of a major cellular cofferdam and subsequently performing substantial modification and rehabilitation to transform the arch dam to a new flat slab configuration, in which difficult foundation conditions and unusual settlement history imposed challenging design constraints, to meet federal requirements and the owner's objectives.

INTRODUCTION

The Niagara Mohawk Power Corporation (Niagara Mohawk) owns/operates 74 hydroelectric sites in upstate New York with a total generating capacity of 680 MW producing some 3,100 GWh of energy annually.

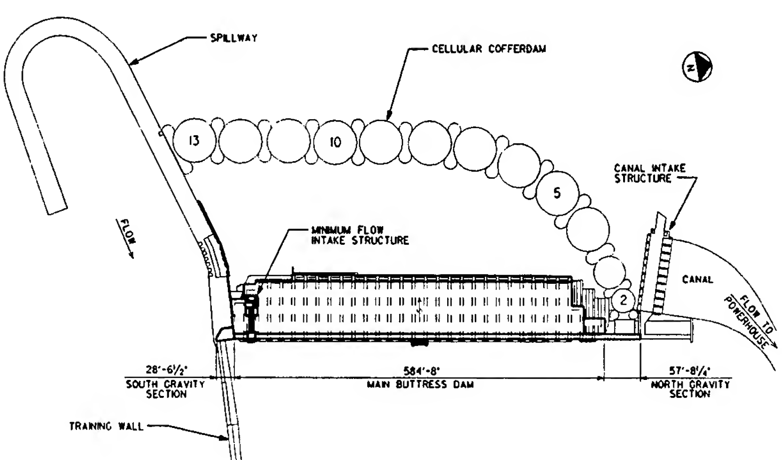
The Sherman Island Hydro Development is located on the Hudson River, about 4.18 km (2.6 miles) southwest of the city of Glens Falls, New York. This development was constructed in the period of 1921 to 1923 by the International Paper Company and was conveyed to Niagara Mohawk on June 30, 1953. Niagara Mohawk later licensed this 28.8 Mw capacity development, along with five other developments, with the Federal Energy Regulatory Commission (FERC) in 1963 as the Hudson River Project No. 2482. The major features of the impounding structures at the Sherman Island Development are illustrated in Figure 1.

[1]Civil Engineer, Niagara Mohawk Power Corp., Syracuse,NY

[2]Project Designer,Niagara Mohawk Power Corp.,Syracuse,NY

PROJECT DESCRIPTION / HISTORY

The dam site is located on the southeastern edge of the central highlands of the Adirondack Mountains. The dam spans a deep, buried gorge comprised of granitic gneisses and schist bedrock. Specifically, the abutments of the dam are founded on granitic bedrock while the central portion of the dam is founded on alluvial sands with varying quantities of gravel and minor amounts of silt. The Standard Penetration Test average "N" value was 30, which is indicative of a medium dense to dense sand. The bedrock overburden is approximately 55 m (180 ft.) deep under the foundation near the center of the dam and decreases in depth fairly uniformly toward either abutment. The dam is immediately founded on a tightly-packed boulder layer approximately 4.6 m (15 feet) in thickness which overlays the alluvial sands and has protected these deposits from scour over the ages.



PLAN

FIGURE 1

The original non-overflow dam consisted of a lightly reinforced concrete multiple arch structure with 31 unreinforced buttresses spaced on 5.8 m (19 ft.) centers. The buttresses were constructed on a 0.9 m (3 ft.) thick concrete slab with shear keys to restrict sliding (Figure 2). The buttress dam is 178 m (585 ft.) in length and has a maximum height of 19.4 m (63.75 ft.) above the base slab. The dam has exhibited settlement and cracking problems from 1923 to 1965. This structure was constructed with no provisions to accommodate movement without cracking.

At its time, Sherman Island had one of the highest concrete dams founded on a soil foundation and elicited considerable interest (Parsons, 1925) in the design of the buttress dam which employed two rows of steel sheet piling at the heel to reduce uplift pressure and eliminate piping of the foundation. Additionally, the arch sections had a relatively flat incline so that the water load would increase the resistance to sliding. Both of these aspects are essential to the stability of the structure.

Structural Settlement

In 1924, after the first year of service, survey instrument monitoring indicated that the dam had settled. The greatest settlement was approximately 1.5 cm (0.60 inches) and occurred near the central section of the dam, coinciding with the deepest sand deposit. Settlement was noted to decrease at and near the rock foundation. This primary consolidation was anticipated but secondary consolidation, which is usually negligible in granular soils, was not expected. However, subsequent years presented continual incremental settlement of the structure, including a modest uniform alignment change in the downstream direction. Between 1923 and 1964 the average rate of settlement was approximately 2 mm (0.08 inch) per year. Since 1964, the average rate of settlement appeared to have decreased to approximately 0.5 mm (0.02 inch) per year. The total settlement to date is approximately 12.7 cm (5.0 inches) at the center of the dam. There was no appreciable differential settlement since differences between adjacent buttresses did not exceed 1.3 cm (0.5 inches).

It became evident that the performance of the dam could not be attributed solely to the immediate, elastic compression of a granular soil, therefore additional settlement mechanisms were investigated (H & A of New York, 1989). After a process of elimination, only two settlement mechanisms appeared to have had the potential for causing the settlement, namely, migration of fine grained soils and cyclic loading.

The settlement-time data indicated that the migration and loss of fine-grained soil may have contributed to the settlement and tilting of the dam. If this was the principal cause, movement of the dam had slowed significantly and the loss of material is considered analogous to the development of a "natural" filter in the foundation soils such that there would be minimal influence in the future.

The sloped surface of the dam confers the relationship whereby small changes in water surface elevation of the reservoir result in relatively large changes in loading to the structure. This leads to the possibility that seasonal and even daily fluctuations in river levels have created the application of cyclic stresses on the soils beneath the dam; stresses that may have contributed to the settlement and tilting of the dam. The reservoir does experience seasonal variations linked to the loss and reinstatement of spillway flashboards and a modest level of pond fluctuation for the facilitation of peak power generation on the upper Hudson River. It has been theorized that cyclic loading has had the most influence on settlement in the period

of 1923 to 1964 and its effect has slowed significantly in recent times.

Structural Deterioration

The dam had experienced concrete deterioration over its service life with various repair measures implemented, including reinforced overlays, shotcreting and grout injection. Pronounced diagonal cracks had developed in each buttress which also displayed deterioration in areas adjacent to the arch sections. The arch sections were extensively deteriorated with the worst sections extending from several feet below the slope change to immediately above the water line (Figure 2). The arches experienced cracking and sections of the concrete had spalled and exposed the reinforcing in the arch in certain cases. The arches continued to leak and freeze-thaw cycles exacerbated the deterioration of the structure.

In an effort to reduce the temperature differential the structure experiences, a masonry curtain wall was constructed spanning between the buttresses in 1956 in an attempt to retain the latent heat of the reservoir. This tended to mediate the previous condition of freezing air contact on the underside of the arches and relatively warmer reservoir waters on the upper side of the arches, thereby lessening thermally induced cracking as a result of wide temperature swings. This slowed deterioration, but over time it became evident that the structural integrity of specific arches was in jeopardy. Leakage was collected and measured in Bay Nos. 9 and 27 and weekly inspections were performed. Given the high risk that an arch would eventually fail, it was proposed that a cofferdam be constructed upstream of the buttress dam to guard against the potential total failure of the dam. The cofferdam would also provide the means for later dewatering of the structure to effect remedial measures.

CELLULAR COFFERDAM

The design, material procurement and installation of the cofferdam was expedited in consideration of the safety concerns at the buttress dam . The cofferdam was designed to satisfy needs for a minimum ten year service life and a two hundred year return frequency design flood. This was considered necessary to allow sufficient time to evaluate rehabilitation alternates, perform the design, provide allowances for the uncertainties of re-licensing the project and eventually construct the remedial measures.

The physical constraints and longevity needs dictated that a cellular type cofferdam be constructed. Steel sheet piles were driven to rock or a minimum elevation of approximately 15 m (50 feet) into overburden to achieve an effective cut-off and lengthened seepage path. The cofferdam was comprised of ten cells, each 20.7 m (67.88 feet) in diameter; one cell, 15.3 m (50.13 feet) in diameter; one cell, 11.5 m (37.60 feet) in diameter; and a braced section. All cells were connected via

arc cells. The cells were installed in an arc extending approximately 244 m (800 feet) from the spillway to the gravity section at the north abutment of the buttress dam. This configuration allowed for the inlet of the power canal to remain unobstructed and continued hydroelectric generation. Construction of the cofferdam was completed in August, 1985 by the Kiewit Eastern Company of Omaha, Nebraska. The total cost for the cofferdam was approximately $10,000,000.

The cofferdam was reinforced with a downstream berm with 2:1 slopes over its largest extent with 1:1 slopes at Cell No. 1 and Cell No. 2 where space was limited on the north end. As a matter of necessity the toe of the berm in these limited cases impinged on the lower arch sections of the buttress dam. Cell Nos. 1, 2 and 3 posed a specific concern as they were founded on rock oriented at an adverse slope for sliding stability. Close monitoring of these cells was a prerequisite to ensure the proper performance of the cofferdam.

The cofferdam was designed for a maximum 15 cm (6 inch) downstream deflection at the top of the sheets and with the stipulation that the cell fill would drain to specified phreatic elevations within the cells. On a cell-by-cell basis, the cofferdam exhibited moderate to no movement. The arching action of Cell Nos. 1 through 5 is the apparent basis for the good performance of Cell Nos. 1 through 3 and validated the stability of these cells that were initially an issue of concern. In practice, each cell must be stable, independent of the influence of adjacent cells. The maximum total movement recorded was in Cell Nos. 8 through 11, with a maximum of 11.4 cm (4.5 inches) downstream deflection.

Worker safety was in the forefront of the rehabilitation construction process and a Temporary Cofferdam Emergency Action Plan that is typically required for FERC projects was developed, along with an automatic safety monitoring and alarm system. A major difficulty in the dam rehabilitation project was the frequent nuisance alarms from this alarm system. The problem was eventually resolved by instituting an intermediate manual intervention step that would screen alarms as to obvious equipment malfunctions. Simultaneously with the analysis of the monitoring system's output, a visual inspection / survey instrument check was performed to dismiss or verify the existence of abnormal conditions that would require orderly evacuation of the work area.

REHABILITATION OF BUTTRESS DAM

Design Objectives and Alternatives

The main objective of the rehabilitation program was to provide for the safe, relatively maintenance-free service life of the buttress dam for at least a 50 year period which is the anticipated term of a new FERC license.

Alternative rehabilitation analyses fell into three main categories: minimal repairs with 3 sub-alternates; significant repairs with 4 sub-alternates and, lastly, total reconstruction (Black & Veatch, 1988). The most extensive of the "significant repair" approaches was selected since it would eliminate all inherent structural deficiencies and be most appropriate for the design objectives for the project (Figures 2 and 3).

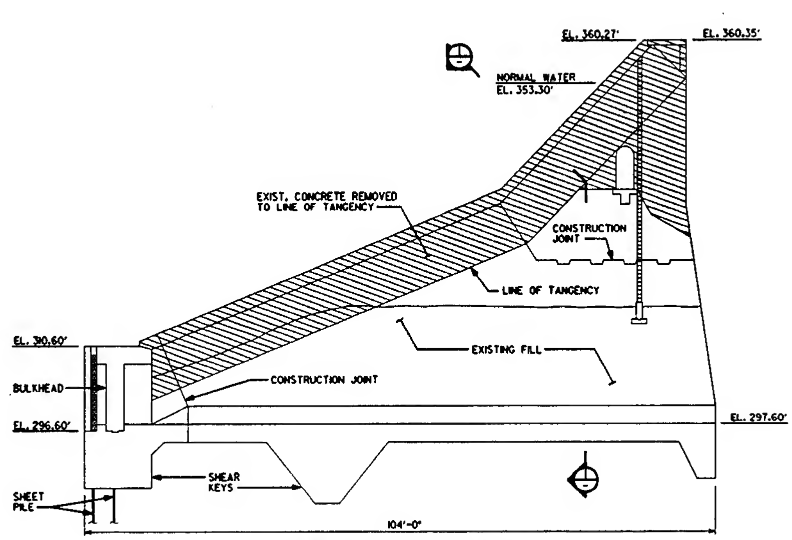
Detailed Investigations

The investigations included the following:

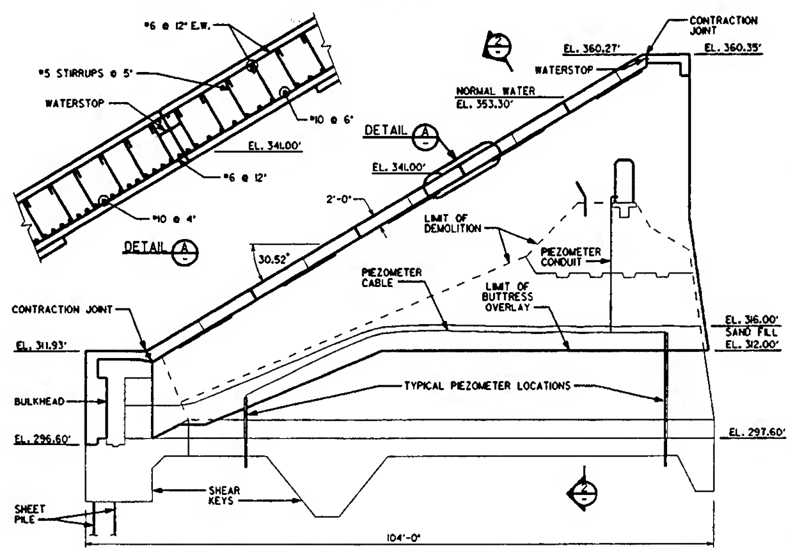
a.) Subsurface investigations including seismic refraction, cone penetrometer testing and subsurface borings.
b.) Removal of backfill within selected bays to determine the depth of deterioration of buttresses and condition of the concrete base slab.
c.) Installation of instrumentation including vibrating wire piezometers, tiltmeters and extensometers.
d.) Testing of existing concrete and mapping of condition of the structure (Base slab, buttresses and bulkheads).
e.) Three dimensional finite element structural analysis of the existing structure and proposed modifications to alter the structure, including the foundation strata.

The reconstructed Sherman Island Dam was analyzed in accordance with the FERC "Engineering Guidelines For The Evaluation Of Hydropower Projects" using Niagara Mohawk's DSTAN computer program for the following conditions:

Case I	Normal Operating Condition
Case II	Normal Operating Condition and Earthquake (Seismic Zone 2)
Case III	Normal Operating Condition and Ice
Case IV	Probable Maximum Flood of 243,000 CFS



TYPICAL SECTION THRU ℄ OF ARCH
ORIGINAL BUTTRESS DAM
FIGURE 2



TYPICAL SECTION THRU ℄ OF BAY
REHABILITATED BUTTRESS DAM
FIGURE 3

Upon completion of the stability analysis, it became apparent that in order to achieve the desired safety factors it would be necessary to further the stability of the proposed structure by imposing additional load on the structure and consequently, the foundation. This was contrary to the recommendations of the geotechnical study (H & A of New York, 1989) that required that the reconstructed structure not impose any additional base pressures and that the existing resultant eccentricities not be exceeded.

In light of this, Niagara Mohawk petitioned the FERC for a variance of the guidelines. In the Case I Normal condition the calculated sliding safety factor was 2.63, which differed from the usual safety factor of 3.0 and in the Case IV PMF condition the calculated sliding safety factor was 1.33 verses the usual of 1.5. After consideration of the various factors, FERC granted the requested variance.

Scope of Work and Design Criteria

The scope of work at Sherman Island consisted of removal of the existing lightly reinforced concrete arches for replacement with new flat reinforced concrete slabs (Figure 4) and removal of deteriorated buttress sections for restoration work with reinforced concrete overlays.

The new reinforced concrete slabs of the Sherman Island Buttress Dam were designed in accordance with the Department of the Army Engineering Technical Letter No.1110-2-XXXX entitled "Engineering and Design, Strength Design Criteria For Reinforced Concrete Hydraulic Structures, Draft" dated January 31, 1990. This guideline superseded ETL-1110-2-312 entitled "Strength and Design Criteria for Reinforced Concrete Hydraulic Structures" dated March 10, 1988.

The existing unreinforced buttresses were reintegrated with an additional 23 cm (9 inch) overlay of reinforced concrete on all surfaces down to approximate elevation 312.0 (Figures 3 & 4). The existing gravity sections at the north and south end were reintegrated with a 30 cm (12 inch) overlay of reinforced concrete. The reintegration of the buttresses and gravity sections was designed in accordance with appropriate sections of ACI-318-89 "Building Code Requirements For Reinforced Concrete" and/or ACI-350R-83 "Concrete Sanitary Engineering Structures".

All structural concrete placed was designed to attain a minimum of 280 kg/cm^2 (4000 psi) compressive strength at 28 days and have a water/cement ratio not to exceed 0.5. Reinforcing steel was in accordance with ASTM A-615,Grade 60 epoxy coated.

Construction

Construction commenced in October of 1990 and was completed in May of 1992. In order to complete the project within the mandated schedule, the general

contractor (Eichleay Corporation, Pittsburgh, Pennsylvania) was required to work a double shift operation for approximately 8 months. Productivity concerns arising from the relatively small work area that was heavily staffed (up to 120 workers on site) were dealt with by closely monitoring the work.

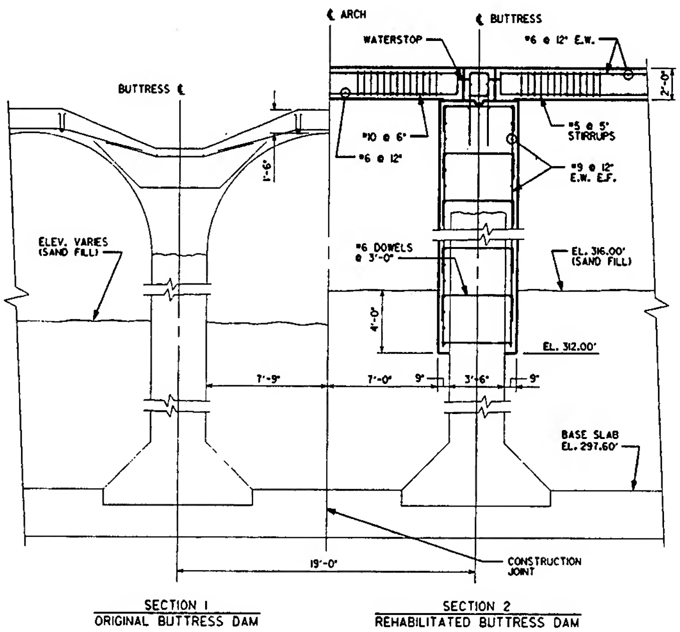


FIGURE 4

The majority of the demolition work was performed using diamond wire sawing instead of conventional percussion demolition techniques in an effort to avoid damage to the remaining structure, particularly the base slab.

The rehabilitation included the installation of approximately 13,770 m^3 (18,000 yd^3) of reinforced concrete, approximately 1,360,000 kg (3,000,000 lbs) of epoxy coated reinforcing steel and approximately 3.5 km (2.2 miles) of PVC waterstop.

The construction quality assurance and inspection program for this project was administered by Niagara Mohawk's in-house Construction Services Department. All daily work activities were monitored by the resident Construction Superintendent and Construction Technicians to insure that the work was completed in accordance with the specifications and drawings and within the schedule restraints.

Commercial testing services were utilized to determine concrete strengths, air content, temperature and slump evaluation.

Field Changes

Renovation work of this nature usually results in a number of field changes. The most significant change involved a redesign of the north end of the structure. In this area, the stabilizing berm of the cellular cofferdam was in close proximity to the upstream face of the existing arches. Concerns about cofferdam stability prevented removal of the berm in order to implement the new configuration. Sheet pile installation was attempted to allow partial removal of the berm, but was abandoned after the risks were evaluated to be too great.

The most cost effective solution to this problem was to fill the last three bays (Nos. 29-31) with concrete. For this to succeed, there had to be assurance that this area was fully founded on rock in order to prevent additional settlement created by the additional weight of the mass concrete. The original as-built drawings indicated that most of the buttresses in the area were founded on rock. A subsurface program confirmed the exact location. It was discovered that Bay Nos. 30 and 31 were entirely founded on rock but that Bay No. 29 was only partially on rock. This condition necessitated recalculation of the stability analysis and re-submission to the FERC for approval. The FERC approved the redesign and part of Bay No. 29 and all of Bay Nos. 30 and 31 were filled with 210 kg/cm^2 (3000 psi) fly ash concrete placed in two foot lifts which successfully reduced the heat of hydration and the risk of cracking.

Instrumentation

The permanent instrumentation system for the buttress dam consists of geotechnical and geodetic components. This system is more sophisticated than monitoring systems normally employed by Niagara Mohawk, but was considered necessary in light of the geologic setting of the structure and its long history of settlement. It is anticipated that the instrumentation will identify trends, by way of the minute differences measured, to determine whether the structure is performing as designed.

The geotechnical monitoring system is comprised of in-place instruments, namely, vibrating wire piezometers to measure the uplift on the base slab; piezometers to measure headwater and tailwater elevations; extensometers to track settlement within the foundation soil mass; uni-axial tiltmeters to track movement trends between survey campaigns; thermistors to determine the temperature of reservoir water, air and concrete; and continuous monitoring via a datalogger with a remote data acquisition system (GZA Geoenvironmental, Inc., 1992).

The geodetic monitoring system allows for the accurate definition of relative structural movement (as measured at the buttresses), since the system identifies and compensates for deformations within the environment (terrain) in which the structure is located (Rohde & Allen, 1992). The survey network is capable of detecting a minimum displacement of $\pm$ 3.0 mm in the horizontal and $\pm$ 0.5 mm in the vertical direction at the 95% confidence level between two survey campaigns. The geodetic system is comprised of electronic theodolites, electronic distance measuring instruments, precision leveling rods, hardened monuments, protected object points, an automatic data acquisition system and a rigorous deformation model which employs statistical techniques.

Performance Evaluation

Rewatering of the area between the dam and the cofferdam was completed over the period of April 27, to May 1, 1992.

Although some leakage was anticipated, no unusual conditions or problems were encountered during refilling. A number of small drips and seeps were noted in the buttress dam upon rewatering. Practically all the bulkheads exhibited some degree of wetness, while the vast majority of slab construction joints did not. A small leak estimated at about 30 ml/sec (0.5 gpm) was noted in the north abutment gravity section. It is theorized that the mass concreting of Bay Nos. 29-31 may have re-directed existing leakage now being experienced at the north abutment. Niagara Mohawk has periodically inspected and recorded the leakage since completion of re-watering and has noted that most of the leaks had dried up completely or slowed considerably. At the time this paper was written (December, 1992), some leakage began to re-appear in localized areas. This is attributed to contraction of the structure as it adjusts to colder temperatures.

It is believed that much of the remaining leakage will seal itself. The general contractor has agreed to assess the leakage at a later time prior to expiration of the warranty and perform any required remedial work in one effort.

In addition to recording the seepage, the status of the structure was checked via the permanent geotechnical monitoring system. Upon rewatering, it was found that the buttress dam displayed little movement and uplift values were well within design values. Data acquisition is ongoing, summarized daily and down-loaded monthly via modem to Niagara Mohawk's office in Syracuse.

The geodetic monitoring system was also utilized to monitor structural movement/deformation. The first monitoring campaign was completed prior to re-watering, the second at the completion of re-watering and the third in late 1992. The results of the second and third campaigns will be compared and will serve as a baseline for future annual survey campaigns.

Costs

The construction cost at completion was approximately $13,000,000 including field changes and additional work performed. This was an approximate 17% increase over the contract award, well within the planned contingency allowance.

CONCLUSIONS

The unique Sherman Island Hydro Buttress Dam was successfully rehabilitated through updating of the original design approach, utilizing selective, low impact, demolition techniques (e.g. diamond wire saw cutting) and reintegrating with reinforced concrete to life extend this structure for a minimum of 50 years in a cost effective manner.

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DODGE FALLS HYDRO - NEW TECHNOLOGY FOR AN OLD SITE

Kenneth A. Oriole, P.E.[1]
A. Sinan Koseatac, P.E., Member ASCE[2]
Mario Finis, P.E.[3]

Abstract

Since the late 1970s, numerous configurations had been investigated for development of the Dodge Falls Hydroelectric Project at a low head site using a 100 year old timber crib dam spanning the Connecticut River between Ryegate, Vermont and Bath, New Hampshire. Originally conceived by the Dodge Falls Hydro Corporation, the Project eventually passed its feasibility analysis in the late 1980's with the selection of an economical layout and design. Development was completed by HYDRA-CO Enterprises, Inc. of Syracuse, New York.

The economical design and arrangement of the Project civil works to complement the high efficiency, low head pit turbine were critical to the successful development of the Project. Some of the factors considered in the design to minimize costs include; documentation of existing site conditions to incorporate as much as possible into the new arrangement; location of the powerhouse for hydraulics, flood control, aesthetics, and constructability; use of existing river contours to maximum benefit; sizing the powerhouse to minimize excavation and concrete while maximizing stability and strength; and designing economical gates for the large water passages.

A highly competent Project team consisting of the Owner, Equipment Supplier, General Contractor, and Engineer, working jointly throughout the Project's construction, and fast track final design of the civil works resulted in the Project being completed on schedule and within budget.

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1 Project Manager, HYDRA-CO Enterprises, Inc. 100 Clinton Square, Suite 400, Syracuse, NY 13202

2 Principal, Northeast Engineering Associates, Inc. 2317 Black Rock Turnpike, Fairfield, CT 06430

3 Project Engineer, Northeast Engineering Associates, Inc. 2317 Black Rock Turnpike, Fairfield, CT 06430

INTRODUCTION

The 5 megawatt Dodge Falls Hydroelectric Project, located on the Connecticut River between Ryegate, Vermont and Bath, New Hampshire, is one of the earliest projects constructed in the United States using very low head with large flows to generate power. The turbine is a fully automatic double regulated Kaplan pit turbine with 5 meter diameter runner rated at 3.7 meters (12 feet) net head. The Project was commissioned in November 1990 and has operated continuously since then meeting its energy projections operating at heads as low as 1.8 meters (6 feet).

The powerhouse is located on the left bank of the Connecticut River about 36 meters (120 feet) downstream from a grouted low height rockfill timber crib dam situated just downstream of a sharp bend in the river and a small island. The powerhouse structure encloses one turbine-generating unit complete with auxiliaries and automatic controls. The power is stepped-up to 34.5 kV in a Project switchyard. Overhead lines carry the power across the river to the right bank tying into a substation located on higher ground. The powerhouse is connected to the timber crib dam by a new concrete gravity overflow spillway.

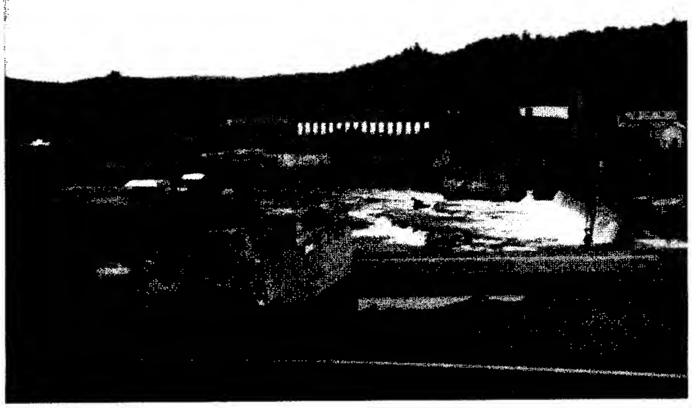

Photo 1 - Overview of Project During Construction

SITE HISTORY AND DESCRIPTION

The site is located on the Connecticut River about 270 miles north of Long Island Sound in a rural landscape of many farms and woodlands. Approaching from the south on New Hampshire Route 135 from the Town of Woodsville, New Hampshire, one must cross a very quaint restored New England wooden covered bridge, which restricted access for construction machinery and large equipment from the south. Along both sides of the river the upper elevations give way to farmland and forest. The New Hampshire shore line was in pristine condition and very scenic consisting of evergreen trees and exposed rock. A sharp bend in the river just upstream of the existing dam presents a scenic outlook when standing on the point. This portion of the river is popular with canoeists and hikers.

The existing timber crib dam was constructed in the early part of this century to provide water to power pulp grinding units in the paper mill on the right bank of the river. The powerhouse portion of the mill acts as a water retaining structure at one end of the dam and is connected to the main mill building running along the river bank. This arrangement is typical of New England paper mills. A training wall at the right end of the dam protects the paper mill from river flow. The turbines for the pulp grinders were removed in 1967 and the draft tubes and intakes were plugged. The turbine pits are currently used by the mill for process water and other purposes. The rockfill timber crib dam was filled with grout in the late 1960s and a centrally located log chute in the dam crest was plugged. The dam averages about 4.6 meters (15 feet) in height.

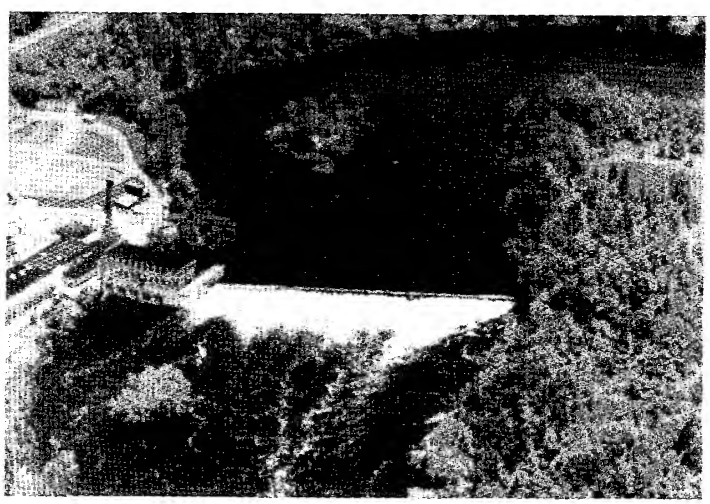

Photo 2 - Overview of Dodge Falls Dam Prior to Construction

Efforts to develop the Dodge Falls Hydroelectric Project were begun in the late 1970s by the Owner of the dam and paper mill, the CPM Company. CPM also owned the land on the left bank of the river upstream and downstream from the dam's left abutment. The only access to this land is through a 100 year old easement over land owned by a dairy farm. The easement, granted in the 1800s, was established for the purpose of constructing the timber crib dam using horses and mules.

In the original FERC License Exemption Application the powerhouse was located on the left bank of the river. The Exemption was granted in May 1982 and subsequently surrendered in early 1984.

In June 1984, the Second Exemption from licensing was granted to the Dodge Falls Hydro Corporation to develop the Project on the right bank of the river using the existing mill powerhouse. Following the 1984 Exemption Application filing, CPM established a relationship with an experienced hydroelectric developer, HYDRA-CO Enterprises, Inc. of Syracuse, New York, to assist in the further development of the Project. The arrangement shown in the June 1984 Exemption Application was found infeasible by the Owner and Developer and they subsequently applied to amend the exemption again to relocate the powerhouse to the left bank of the river. This arrangement was granted an exemption and is the arrangement ultimately constructed.

During this time, approval of a power purchase agreement was being pursued that would sell the Project's energy to Vermont Retail Utilities through the Vermont Power Exchange, a purchasing agent. This approval came in June 1988 accelerating Project development.

In the early 1980s, double regulated horizontal turbines were common in the small hydro industry in the United States. The first exemption application was based on three horizontal S-Type Kaplan turbines in a low profile powerhouse which would be overtopped at less than the 100 year return period flood. The powerhouse was connected to an entrance building located at higher ground by a cut and cover tunnel. The cost of three water-to-wire turbine-generators with all appurtenant equipment, a large powerhouse for the units, maintenance equipment, gates, and the cofferdams required to enclose the work area were increasing the cost of the Project and hindering feasibility.

The second exemption application used the old powerhouse on the Vermont side of the river, but the area for the units that were removed was now occupied by pollution control equipment, cooling and process water pumps, and other mill equipment. To develop the hydro potential of the site using the existing mill powerhouse this equipment would have to be relocated and a place would have to be found for this equipment in an already crowded plant, interrupting the operation of the mill. The existing openings would then have to be modified to receive the horizontal S-Type Kaplan turbines requiring deep excavation in the mill foundation.

The intake, trashracks, and gates would all need renewing and a new set of gate guides would be needed at the tailrace. Access to the powerhouse was available only through a plant parking lot and loading area or through the existing mill. Three separate access easements would have to be obtained just to access the dam from the Vermont side of the river and to construct the transmission line. Because of the restricted access, delivery and installation of the turbine-generator equipment was a costly proposition resulting in this arrangement being found infeasible. In addition, the mill was considered a historic landmark which imposed additional restrictions on Project development.

The Amendment to the second exemption application proposed moving the powerhouse to the left bank of the river, the New Hampshire side, as originally envisioned. This time the proposed powerhouse would contain one large unit. During the mid-1980s, axial flow open pit turbines started to gain acceptance in the small hydro industry, although bulb turbines had been available for some time. Similar units had been successfully installed on the Allegheny River in Pennsylvania and in Sault Sue St. Marie in Canada.

The Owner, using the License Exemption Amendment drawings and the proposed equipment arrangement, received quotations from various Equipment Suppliers and General Contractors for the major work items in the Project. Before the decision by HYDRA-CO finally settling on a single unit installation on the New Hampshire side of the river, some of the suggestions made by equipment suppliers included vertical turbines in the mill powerhouse, submersible turbine-generators downstream of the dam, and a new powerhouse next to the existing mill powerhouse downstream of the dam. These alternatives were considered but were rejected for a variety of reasons including environmental impacts, construction cost, access, and long construction times, among others.

In early 1988, new estimates for Project cost and energy based on a single pit turbine were prepared which resulted in a financially attractive project. The Project team, consisting of the Engineer, Equipment Supplier and General Contractor, was selected in mid-1988 from a competitive bidding process. Contract level drawings were begun in mid-August in parallel with equipment engineering to obtain further savings. The General Contractor, Engineer, and Owner participated in a value engineering analysis to arrive at a constructible scheme while controlling costs. Construction of the facility was contracted for under a lump sum agreement and construction finally began in May 1989. Commissioning of the equipment started in October 1990 and commercial operation began on December 1, 1990.

CRITERIA FOR PROJECT DESIGN

With the decision made to develop the Project on the New Hampshire side of the river, the Vermont agencies were concerned about the aesthetics to people standing on the Vermont bank looking towards

New Hampshire. The Vermont Public Service Order that approved the Power Purchase Agreement called for the Developer to prepare and submit a rendering depicting the powerhouse and site features set in the New Hampshire shore line as viewed from the Vermont shore. To satisfy the Vermont agencies on aesthetics issues, a number of suggestions were made by the Vermont Board's architectural consultant including: constructing the powerhouse to complement the brick and concrete features of the paper mill; releasing 3 inches of water over the dam for aesthetics; and consider an alternate access road through a ravine on the downstream side of the powerhouse.

The developers had to defend their position on each issue in a hearing before the Board. As a result, the alternate access road route was the only suggestion enforced by the Board. Photographs of the New Hampshire bank were prepared with images of the powerhouse and the alternate access road superimposed on the photographs and the Board's architectural consultant was invited to view the bank again during construction. The developer had concerns about the safety of the steep slopes required by the alternate route and a desire to leave the ravine in its natural state since the ravine was home to many evergreen trees and had scenic and habitat value. After reviewing the situation and considering the other provisions being made, such as architectural treatment of the powerhouse and choosing natural boulders instead of steel guardrails, the Board reversed its decision.

The powerhouse is located downstream from a sharp bend and small island (Marshall Island) in the river channel. River cross sections were taken from the dam to upstream of the bend showing that the depth of water at the upstream end of Marshall Island in the river bend to be over 6.1 meters (20 feet) and showing the river channel being shallow on each side of the Island. The flow area gets larger at the downstream end of the Island.

The flow in the river approaches the bend uniformly, then gains momentum on each side of the island. The challenge in designing the new powerhouse located on the inside bank of the river bend was to capture the full river flow with a single unit intake without excessive headloss in the intake channel. Since the head available for the Project is in the range of 1.8 to 4.9 meters (6 to 16 feet), an inefficient approach channel and intake would result in a high percentage of energy loss.

A solution was found by hydraulically streamlining the intake and tailrace channel using the cross section information. The design of the intake approach channel allows the turbine to capture the full river flow from each side of Marshall Island.

A requirement for a 0.61 meter/second (2 fps) intake approach velocity to minimize fish entrainment, and a good unit price for rock excavation early in the Project allowed the Engineer to design a hydraulically efficient and cost effective intake

approach channel. Although improving the hydraulic conditions in the approach channel, the limitations on approach velocity required a wider power intake and larger trashrack area which could have increased costs. The wider intake was used to improve the stability of the powerhouse, and the trashrack panels were installed in face mounted guides which required only anchor plates be embedded. Through innovative solutions, these requirements were incorporated and allowed lower costs and faster construction.

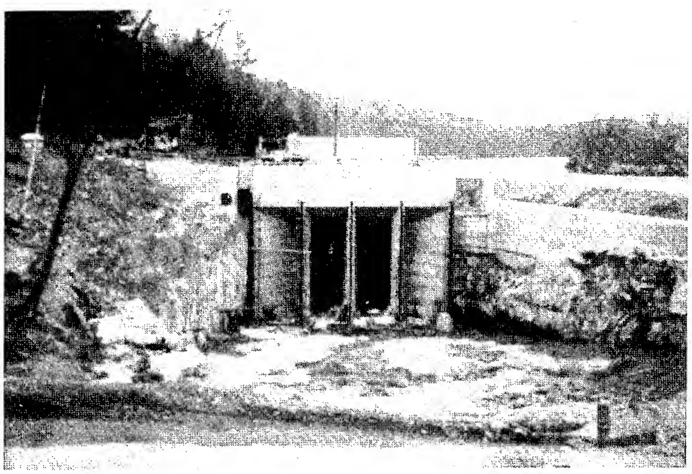

Photo 3 - Project Headrace and Intake

A trashrack rake with telescopic boom manufactured by the Alpine Machine Company is used to service the vertical trashracks. An advantage of this rake is the ability to locate the electrical equipment high in the frame providing protection from flooding and allowing the intake deck to be set below the 100 year flood level.

Two intake roller gates close off the large water passage on each side of the pit structure. Each gate was manufactured in three pieces and bolted together in the field and is provided with its own electric hoist. The gates are designed to close by gravity in an emergency situation. The gate scheme using dedicated electric hoists and support structures was found to be more cost effective than hydraulic systems. Another advantage is the ability to locate the hoists on the framing above the 100 year flood level.

The guides for the draft tube slide gate are located as close as possible to the centerline of the runner to reduce the span and cost without affecting the turbine equipment performance guarantees. The gate is also designed and manufactured with six sides to reduce the headlosses from the guide slots and to self clean and prevent silting of the slots.

The weir gate on the river side of the intake serves several purposes. On unit shutdown the gate automatically opens and discharges a minimum environmental flow release to the downstream side of the dam, keeping the headwater level from rising quickly thereby minimizing the frequency of flashboard collapse. The gate is also used as an ice sluice and a downstream fish passing device together with a smolt bypass box.

The Project's intake deck is set below the 100 year flood elevation, but the powerhouse roof and Project switchyard are set well above this level. The site last experienced an estimated 100 year flood, (roughly 1650 m^3/s or 58,000 cfs) in 1936. The air inlet louvers and main entrance door are located in the downstream wall of the powerhouse in case of future flood flows around the powerhouse. The main entrance door is water tight and designed to take 0.61 meter (2 feet) of water pressure. The low point of the louvers is set above the 100 year flood level.

The unmanned plant uses automatic controls with an emergency generator and an automatic transfer switch for backup power. The emergency generator supplies power to the sump pumps, lube oil and cooling water pumps, battery charger, lighting transformer, and gate hoist motors in an emergency. Submersible drainage and dewatering pumps are sized to operate at full capacity with the river at 100 year flood levels. The turbine control system operates either automatically or manually and incorporates a digital governor in conjunction with a PLC.

During the development and design stage, one of the problems faced was the interconnection of the Dodge Falls Project to the Utility. The nearest power lines on the New Hampshire side of the River were quite a distance from the Project Site and would have required negotiating easements through numerous properties. Across the river from the Dodge Falls station in Vermont was a substation that could serve as the interconnection point with Green Mountain Power Company of Burlington, Vermont. Because the location of the interconnection point was in a different state than the plant substation, redundant disconnect and protection features were required by the host utility so that utility crews would not have to leave the state.

Because the closest river crossing bridges were four miles from the Project site in either direction, the distance between the plant's powerhouse and the point of metering and utility interconnection was approximately ten miles. To get from the plant substation to the interconnection point, the transmission line had to pass over the River and through the mill property. Since the Vermont bank contained roads for the mill, settling ponds, and other mill facilities, a route had to be chosen considering these facilities and span distances, easements and costs. In the final analysis the Project was connected to the grid in Vermont by a 34.5 kV overhead line after stepping up generator voltage in its own switchyard on the New Hampshire side.

CONSTRUCTION

The General Contractor, Pizzagalli Construction Company of South Burlington, Vermont, mobilized in May 1989 and, after construction of the access road, immediately began work on the upstream and downstream cofferdams. The historic average daily flow in the river was 139 m^3/s (4915 cfs). Because of the upstream storage reservoir on the river, the utility plants upstream from Dodge Falls are operated as peaking plants and flows of 170 m^3/s (6000 cfs) or greater could easily be experienced daily at the Dodge Falls dam. A tributary to the Connecticut River joins the river downstream of the Comerford Dam and reservoir adding to the daily flow releases from the upstream peaking plants. As a result of these conditions the flow at the site could very quickly reach 280 m^3/s (10,000 cfs) in the event of any precipitation in the tributary's drainage basin.

The unpredictability of the river was evidenced in March of 1990 after a week of record warm temperatures in the Northeast caused a massive snow melt. A major flood resulted and breached the Project cofferdams and flooded the work area. Fortunately at that time the powerhouse walls were up and the only pieces of equipment in the powerhouse were the pit structure and draft tube liner. The flood and clean-up delayed construction about six weeks.

The General Contractor chose to obtain the formwork for the large and complex water passages from an off-site fabricator rather than construct them on site. During fabrication the form manufacturer caught a minor design flaw when they created three dimensional images on computers. This flaw could have been a major delay in the construction schedule and might have caused a great deal of change order requests had it not been caught at this stage. By prefabricating these forms indoors under controlled conditions, over a month of schedule time was saved.

The major equipment pieces were manufactured at various locations around the world. The runner blades and hub came from Europe, the pit structure, and draft tube liner were manufactured in Canada, and the distributor ring, wicket gates, and inner and outer rings came from Mexico. As a result, the turbine parts met each other for the first time at the site, and the absence of shop assembly created some minor difficulties.

All the major mechanical components, including the turbine, were installed successfully meeting very exacting tolerances by the General Contractor under the supervision of the Equipment Supplier. The distributor ring installation was a complex operation in itself. Brainstorming sessions were held as to how best to install the ring due to its large size and great weight. The result was a special crane being mobilized and the distributor assembly being lowered into the turbine pit in one lift.

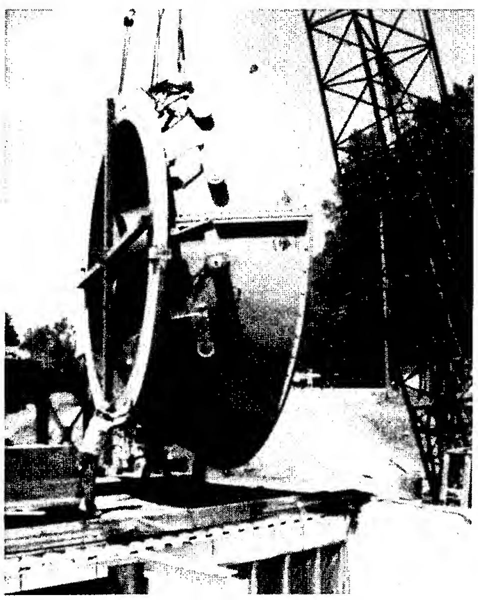

Photo 4 - Installation of Discharge Ring Assembly

CONCLUSIONS

The Dodge Falls Project is a living example of applying modern technology to utilize an existing site to produce energy with virtually no additional environmental impacts while preserving a historical landmark.

Persistence and repeated attempts to apply value engineering principles were two necessary ingredients required to bring this Project from concept to reality. The Project also demonstrates the effectiveness of teamwork as Owner, Contractor, Equipment Supplier and Engineer all work together to overcome obstacles toward the common goal of completing the Project on-time, within budget, and achieving the required technical objective.

ACKNOWLEDGMENTS

The writers wish to acknowledge the cooperation of the Vermont Power Exchange, Vermont Public Service Board, Green Mountain Power Corporation, Vermont Independent Power Producers Association, Vermont and New Hampshire Public Agencies, Shawmut Bank, and the Project Owner, HCE-Dodge Falls, a subsidiary of HYDRA-CO Enterprises, Inc., and the John Hancock Mutual Life Insurance Co., in addition to the Project team who contributed to the successful completion of the Project.

Turbine Improvements at the Ford Hydroelectric Project

John Rohlf[1] and George Waldow[2], M. ASCE

Abstract

The 17.8 MW Ford Hydroelectric Project was constructed on the Mississippi River in 1924 to provide power for a new vehicle assembly plant in St. Paul. After nearly 70 years of reliable service, the four original Francis turbines were clearly in need of rehabilitation. Coincidentally, the Ford assembly plant was experiencing significant load growth which the hydro facility could not accommodate. Studies indicated that project output could be increased substantially through installation of new state-of-the-art turbine runners. In addition, the existing generators had been rewound and could handle the additional turbine horsepower. After securing a FERC license amendment, two units were selected for the initial upgrade contract. Testing of the first completed unit revealed a 40 percent improvement in peak output.

The methods utilized and the lessons learned on this project will be of interest to others who are contemplating the best approach to rehabilitate and improve production at an ageing hydropower facility.

History

Ford Motor Company has owned and operated a hydroelectric project on the Mississippi River in St. Paul, Minnesota for 70 years. The project has been a

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[1]Engineering Supervisor, Twin Cities Assembly, Ford Motor Company, St. Paul, Minnesota

[2]Business Development Project Manager, Stanley Consultants, Inc., Minneapolis, Minnesota

reliable source of low cost electricity to the Twin Cities Assembly Plant. Excess power is sold to Northern States Power Company for distribution. The powerhouse was constructed by Ford in 1924 and contained four Wellman-Seaver-Morgan vertical Francis turbines, each rated at 4,500 HP at 34 feet of head. The 100 RPM units are directly coupled to Westinghouse generators, each rated at 4,500 KVA. The facility is manually operated and staffed 24 hours a day, seven days a week. All equipment maintenance is performed by Ford personnel. Civil and structure maintenance is performed by local contractors. The only major work done on the generation equipment was rewinding of the generators in 1968. Various changes and upgrades to the distribution switchgear have occurred throughout the years.

A 574-foot long Amberson-type dam connects the powerhouse with navigation locks at the opposite end. Lock and Dam No. 1 is owned, operated and maintained by the Corps of Engineers except for the sluice gates and flashboards which are operated and maintained by Ford.

Upgrade Justification

A major expansion of the vehicle assembly plant in 1985 resulted in peak electrical demand exceeding peak hydro generation capacity for the first time in history. Engineering studies were initiated to identify cost justifiable projects that would increase internal generation capacity. The two major projects considered were upgrading the hydro facility and installation of a cogeneration unit. The hydro upgrade concept considered several alternatives including: rehabilitation of the existing components; installation of either two or four new Francis runners; installation of a new vertical Kaplan unit in one of the existing bays; and installation of an additional turbine in the small exciter bay. A preliminary benefit cost study clearly indicated new runners to be the best hydropower option.

The existing generators had been rewound in the late 1960's and were believed capable of accepting 6,500 HP with minor modifications. With this as an upper limit, other items such as shaft integrity and draft tube limitations were analyzed. Minimizing the amount of concrete work was desired due to limited available funding.

Based on engineering analysis and further input from turbine suppliers, it became evident that upgrading even two units to 6,500 HP was cost prohibitive. In addition

to draft tube limitations, there was some doubt as to whether the generators could be reliably operated beyond their rated capacity. Ford therefore reconsidered and decided to limit turbine output to 6,000 HP.

The final hydro upgrade proposal consisted of replacing two of the original cast iron runners with 6,000 HP stainless steel runners. The cogeneration proposal and the final hydro proposal were then internally evaluated for funding approval. The areas evaluated included economic payback, future maintenance, permitting and daily operational requirements. Based on comparative analysis the hydro upgrade project was selected for implementation.

The comprehensive economic payback analysis was required due to the size of the expenditure. Annual capital budgets are typically far short of what would be required of a project of this magnitude. The analysis was based on a time adjusted rate of return, affected by inflation, depreciation, return and investment. The economic benchmark for internal project approval continually changes based on Company conditions. In very rough terms, the upgrade project needed to pay for itself in three years. Upgrading only two of the four runners initially was all that could be cost justified. This was due to the rigorous economic evaluation criteria and progressive impact of declining water availibility and unit utilization.

FERC License Amendment

The agency consultation process was initiated to pursue a FERC license amendment for increased plant capacity. The project had been re-licensed in 1980 and the objective was to get an amendment to upgrade all four units; two immediately and two in the late 1990's.

For reasons not known, the original rated turbine capacity was substantially less than rated generator capacity. In addition, the turbines were rated at "best" gate while the licensed plant ratings for hydraulic capacity and installed capacity were based on maximum (full gate) output. The replacement runners were sized to match, but not exceed, generator nameplate capacity as indicated on the following page.

	Original Units	Upgraded Units
Rated Turbine Power (best gate) at 34 ft Head	4,500 HP	6,000 HP
Maximum Turbine Power (full gate) at 34 ft Head	4,900 HP	6,000 HP
Maximum Turbine Discharge at 34 ft Head	1,600 cfs	1,750 cfs
Generator Rating	4,500 KVA	4,500 KVA

The project concept would increase plant hydraulic capacity by 600 cfs. This additional flow is available nearly 40 percent of the time and has historically been spilled over the dam. Utilization of this "surplus water" during high flow conditions presented no significant environmental impact.

FERC defines plant capacity as the sum of generator nameplate capacities. The 6,000 HP runners would not constitute an increase in plant capacity with respect to the project license since rated turbine capacity did not exceed rated generator capacity. In addition, Ford was already being assessed an annual charge based upon generator nameplate capacity.

Since the proposed unit modifications involved no new capacity and were not substantial in scope or probable impact, FERC ruled that the amendment process could consist of a single consultation stage. In addition, agency consultation was to focus exclusively on proposed changes and new or incremental impacts; the process was not to address or reopen issues related to the existing licensed facility.

The primary consultation issues involved convincing review agency staff that the proposed runner changes would have no impact during low flow conditions and that the project would continue to operate in a run-of-river mode.

The non-capacity license amendment was subsequently issued in a timely manner.

Equipment Selection Process

A specification package was prepared for competitive bids. The base scope of work included the design, fabrication and complete installation of two 6,000 HP stainless runners. Knowing that many other components could be in need of attention, the suppliers were asked to provide an itemized price list for replacing various turbine components. A dewatered inspection of one of the units was conducted during the prebid meeting.

All bids received exceeded available project funding. Two of the suppliers were then requested to submit cost reduction proposals. Both suppliers approached the cost reduction process differently; however both lowered their unit output guarantees. Items such as daily clean-up, crane service for shipping, and miscellaneous painting were deleted from the scope of work or assumed by Ford. Normally the low qualified bid receives the award. In this case, with independent approaches to reduce cost, the final decision was based on a ratio of guaranteed unit output versus total unit cost. The contract was ultimately awarded to Voith Hydro, Inc. of York, Pennsylvania.

Unforeseen Problems

The basis of the project was cost justified turbine runner upgrades. There was very little funding reserved to address problems with the other unit components. The scope did include turning the wicket gate stems, line boring the head cover and bottom ring, and installing new greaseless bushings for the wicket gates.

The wicket gates themselves offered the biggest surprise. Clearances found in the lower bushings were 0.10"; original design clearances were 0.002". Also, the gate stems had extensive sub-surface cracks, many of which were half way through the stem. The hollow gates also had cracking on the vertical surfaces of the leaf along the sealing areas. After close comparison with the original design prints, it was discovered that the gates had been cast upside down. The two masses of material to support the stems were reversed, with the large portion at the bottom for the short bottom ring stem, and the small portion at the top of the gate. After lengthy discussions, it was decided to cut off the old stems and replace them with new stainless stems. Costs for this undertaking approached 40 percent of the original contract amount. Additional funding for this work was

difficult to obtain and put the project on hold for several weeks.

Another surprise was the status of unit alignment. In both units, the stator had to be moved to bring the system into final alignment. There was no evidence of shifting, only speculation about the original installation. Non-symmetrical bearing wear had been observed on one of the units during maintenance.

The item which was potentially the most devastating was the existence of only a single set of head gates. Knowing that the one set of bulkhead type gates were going to be dedicated to the first unit rehab for several months, the other units were dewatered and cleaned out. The trash racks were also checked for integrity.

After the first unit was disassembled and the headcover sent out for machining, a problem arose on one of the other units. The wicket gates would not close. This had never happened in the history of the project. There was a "clicking" sound from the headcover which appeared to indicate something was lodged in the runner. Attempts to identify the single stuck wicket gate resulted in 6 of the 16 gate links breaking in rapid succession and rotating over center. The head gates could not be removed from the first unit because it was completely disassembled. The problem unit was on line at 95 percent gate and performing fine. There was real concern that the unit would trip off line and go to runaway. To further complicate the situation, control wiring changes from years past eliminated the ability to stop excitation by opening a switch. A runaway unit would still have excitation!

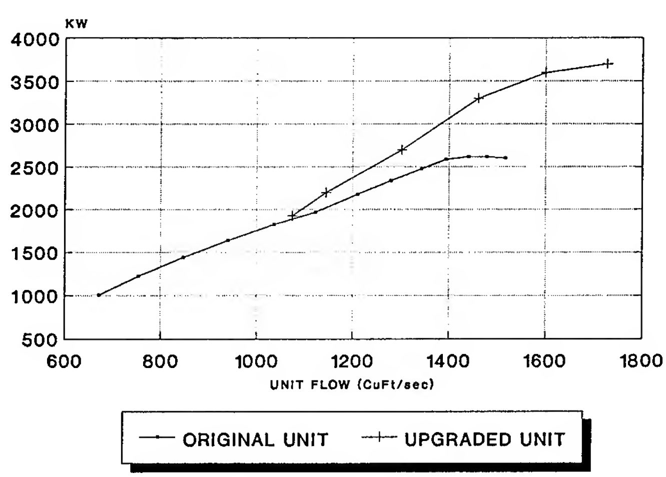
The solution to stopping the water flow was to fabricate and install a new set of head gates. The original two gates are 14' wide and 22' high. The design, fabrication and installation of the new gates was accomplished in just sixty hours. Special thanks went out to the local steel fabricator, Ford accounting/purchasing and especially the Ford maintenance personnel for this feat. To overcome installation problems during full flow, the new gates were made in sections. Eight sections were fabricated (4 per bay) and then installed one at a time. Each section was only 5.5' high. This allowed the water pressure to equalize before sliding the gate down into position. The last gate required hydraulic jacks to push it down into final position. This was done by supporting an I-beam across the normal head gate support stations and jacking down.

The flow was reduced to extensive leakage and the unit went into a motoring condition. Sand bags and cinders were then used to reduce a portion of the leaks. The wicket gates were closed and the unit came to a stop. The clicking sound stopped as soon as the wicket gates moved. At this point, the leaks were still greater than what the drain could handle, so the head gates were removed and reworked. The sections were bolted together and the seams caulked. Eventually the leaks were completely stopped and the unit was successfully dewatered. Inspection of the runner revealed a log which was 6" in diameter lodged between two buckets. This unit was then chosen as the next unit to be upgraded.

<u>Results</u>

Output of the original unit was carefully measured at different gate settings using existing plant metering equipment. Similar measurements were then made with the new runner in place. Comparative performance results (at available test head) are shown below:

FORD HYDRO RUNNER UPGRADE
GENERATION VS. WATER FLOW



32.4' EFF. HEAD

The deteriorated condition of the original unit was obvious; it was performing well below design levels. Maximum KW output increased by over 41 percent with the new runner. Unit efficiency also increased significantly throughout normal operating range of the unit. Predicted performance objectives were achieved. Based on the substantial measured improvements, Ford opted not to conduct more rigorous and expensive testing procedures.

Conclusion

Ford's experience demonstrates what can be encountered when an ageing hydropower project is upgraded.

Industrial Manufactures are primarily focused on their production activity, be it automobiles, paper or cement. Nonproduction activities such as rehabilitation or upgrade of hydro plants typically receive less priority and capital funding. Therefore, creative approaches which demonstrate short-term payback are required to obtain project funding.

Substantial turbine performance improvements are possible with current technology. Turbine suppliers can reliably estimate incremental generation benefits which can then be used for project justification analysis.

When undertaking unit rehabilitations, owners should be prepared to encounter unforeseen problems, and should have procedures in place to react. Projects of this type are rarely completed without a few surprises. The obstacles should not prevent industrial owners from pursuing hydropower upgrades. The economic and environmental benefits obtainable by optimizing this renewable resource far outweigh the challenges presented.

CABINET GORGE ARCH DAM FINITE ELEMENT ANALYSIS

Marc Van Patten[1]
Mike Pavone[2]
Steve Benson,[3] Member ASCE

Abstract

As a result of the 1987 Part 12 inspection report, a material investigation and stress analysis were performed for the Cabinet Gorge Project. Subsequent to the material investigation and analysis, Washington Water Power (WWP) received a letter from the FERC commenting on the work that had been performed. In the FERC's letter they noted several items in the previous analysis that had either not been considered or were not in keeping with the FERC "Guidelines for the Evaluation of Hydropower Projects." The FERC commented on the elastic properties of the concrete and foundation materials, the foundation shear parameters, the need for a cracked base analysis, and foundation drainage. The analysis contained herein made use of the latest features available in finite element analysis to resolve the concerns raised by the FERC and to bring the issue of stress and stability of Cabinet Gorge Dam to a close.

Introduction

The Cabinet Gorge Dam is located on the Clark Fork River in North Idaho, just west of the Idaho-Montana border. Cabinet Gorge Dam is a concrete, double curvature arch dam positioned between two concrete thrust blocks. The arch dam has a crest length of 120 m (395 ft) and is 63.4-

[1] Principal Engineer, Ebasco Infrastructure, 10900 NE 8th Street, Bellevue, WA 98004-4405

[2] Consulting Engineer and Manager of Planning and Geotechnical Dept., Ebasco Infrastructure, 10900 NE 8th Street, Bellevue, WA 98004-4405

[3] Principal Engineer, Ebasco Infrastructure, 10900 NE 8th Street, Bellevue, WA 98004-4405

meters-high (208 ft) at its tallest section and 13.7-meters-wide (45 ft) at its thickest section. The arch dam has a gated spillway section which rests along the entire crest of the arch. The right abutment is a short gravity section with a trash sluice way. The left abutment is a large concrete gravity thrust block section which is integral with a smaller wing dam section. See Figure 1.

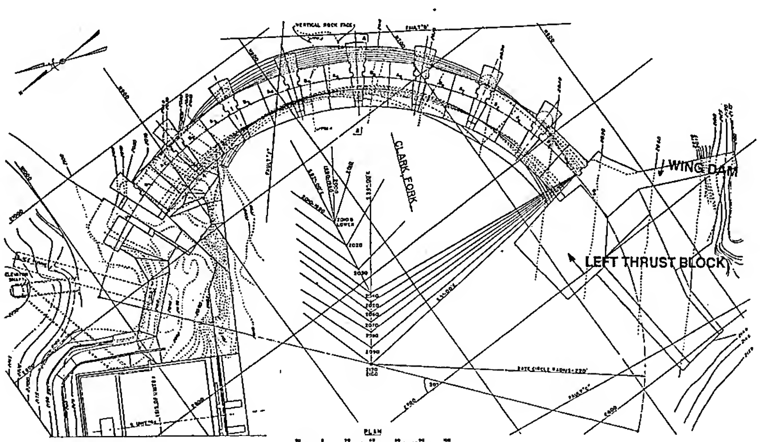


Figure 1. Plan View of Cabinet Gorge Dam

A finite element analysis of the arch dam and thrust blocks was performed as part of the 1992 Part 12 safety inspection report. The analysis was aimed at satisfying the FERC's concerns with regard to elastic and shear material properties used in the previous analysis, tension at the dam/foundation interface, tension at the arch/thrust block interface, and uplift assumptions. Refinements to the analysis included using measured reservoir and tailwater temperatures, performing a heat of hydration evaluation to verify closure temperatures, performing a full 3D thermal transient analysis for input to the stress analysis, providing vertical construction joints in the arch dam model, providing a compression only contact surface between the arch dam and foundation as well as between the thrust blocks and foundation, applying uplift pressures to the entire base of the arch and gravity sections of the model, applying ice loads with the winter load case, using improved concrete and foundation material properties, and performing a construction sequencing of the gravity loads and joint grouting in the arch dam.

Most of the improvements mentioned above are now possible due to recent improvements in the field of finite element analysis. One significant

improvement in the finite element model was the use of contact surface elements at the contact between dissimilar adjacent sections of the model. With contact surface elements it was possible to model a compression-only contact as well as sliding friction along a contact with two dissimilar meshes. A similar effect could be modeled in the past using gap elements; however, gap elements require the mesh along the contact to be identical and are sometimes difficult to include in a large three dimensional model. In addition, many finite element programs do not offer the capability of modeling more realistic non-linear effects. For this reason, many finite element analysts have chosen to model arch dams using linear modeling techniques. Linear modeling techniques can sometimes result in substantial modeling errors which could ultimately produce answers which do not accurately represent the loading characteristics of the dam. Such errors could be in the form of high tensile stresses at the foundation contact, which in actuality do not exist. Allowing these high contact tensile stresses to occur may have a significant affect on the distribution of stresses throughout the dam and adjacent dam sections. For Cabinet Gorge Arch Dam, modeling the compression-only contact surfaces at the base of the dam made a significant difference in both the stresses in the arch portion of the dam as well as in the thrust loads carried by the left thrust block and wing dam sections.

Another improvement to the model was the use of birth and death. This technique allows the modeling of adjacent arch dam blocks as it was actually constructed in the field. With this technique, it is also possible to simulate the affect of having the joints between blocks grouted in different stages. This will produce a more realistic application of gravity loads into the foundation as well as subsequent arch loads into the abutments.

Stress and Stability Evaluation Criteria

1. Arch Dam

The arch portion of the dam was evaluated for internal stresses, both in the cantilever and arch direction. Computed stresses were compared to the allowable stresses for each load case, for both tension and compression. Contact stresses between the arch and rock foundation were also compared against the allowable compressive stress of the concrete. Finally, computed radial displacements of the arch dam were compared to actual measured values.

In order for tensile stresses not to develop along the foundation of the arch dam, special elements were used to allow the arch dam to act independently of the foundation with respect to tension. This allowed for the

arch dam to move freely with the applied loads and for the arch dam to redistribute the loads as necessary.

2. Left Thrust Block and Wing Dam

The left thrust block and wing dam sections are tied together with extensive reinforcing and was therefore analyzed as an integral unit acting monolithically. Given the presence of keys between the arch dam and the thrust block, shear and compressive stresses were allowed to be transmitted across the joint; tensile stresses were not. The monolith was investigated for stress and stability as follows:

a. Internal concrete tensile and compressive stresses were computed for the various load cases and compared to the allowable stresses.

b. Foundation bearing stresses normal to the concrete/rock interface were compared to the allowable compressive stresses of the concrete (the weaker of the two materials) for the respective load cases.

c. The foundation contact area in compression was evaluated to ascertain whether cracked base conditions would apply.

d. The issue of sliding potential along the concrete/rock interface was addressed giving consideration to the plausibility of the sliding scenario. For example, if the full magnitude and direction of the thrust vector acting on the monolith was oriented in such a direction that it would tend to drive the block into the abutment rock behind the block, that would not constitute a plausible sliding scenario. Therefore, the only plausible scenario where sliding along the concrete/rock interface could occur would be to the north, towards the river.

3. Right Thrust Block

The right thrust block is favorably oriented to resist the dam-imposed thrust loads and was therefore not evaluated for sliding stability. The thrust block was evaluated for base concrete/rock interface pressures versus the allowable concrete compressive stress.

<u>Analysis Criteria</u>

1. Loads

Loads consisted of hydrostatic loads, uplift, dead loads, ice loads, and temperature loads. The headwater and tailwater levels for the Normal Load Cases were 662.9 m (2175.0 ft) and 630.3 m (2068.0 ft), respectively. The headwater and tailwater levels for the Flood Load Case were 665.7 m

(2184.0 ft) and 644.3 m (2114.0 ft), respectively. The unit weight of water was taken as 1000 kg/m^3 (62.4 pcf). Uplift was assumed to act along the entire foundation interface. Uplift was assumed to vary linearly from headwater pressure at the heel to tailwater pressure at the toe for the arch dam section. Where a tension-induced separation was computed at the dam/foundation interface, for reservoir levels where piezometer data does not exist, full reservoir pressure was introduced into the crack, decreasing linearly to tailwater pressure at the toe. Dead loads consists of the weight of the arch dam, gates, piers, and thrust blocks using a concrete unit weight of 2,371 kg/m^3 (148 pcf). Ice loading was taken as 73 kN/m (5 kips/ft) over the entire upstream side of the dam and thrust blocks acting at the normal headwater level of 662.9 m (2175.0 ft). The ice load was only included in the Normal Winter Load Case.

The thermal transient analysis of the dam incorporated two annual cycles of seasonal temperature changes, which included both air and measured water temperatures. The result of the thermal analysis produced annual cycles of concrete temperatures within the arch dam and thrust block sections. From these annual cycles of concrete temperatures, the maximum and minimum concrete temperatures were taken for inclusion in the stress analysis for the summer and winter load cases, respectively.

Based on construction records and a heat of hydration analysis, two closure temperatures were used in the analysis of the dam. Below El. 637 m (2090 ft), the closure temperature was taken as 12.8 degrees Celsius (55°F). Above El. 637 m (2090 ft), the closure temperature was taken as 18.3 degrees Celsius (65°F).

2. Load Combinations

Usual Loads:

a. Normal Operating plus Spring/Fall Thermal Loads
b. Normal Operating plus Winter Thermal and Ice Loads
c. Normal Operating plus Summer Thermal Loads

Unusual Loads:

d. PMF plus Spring/Fall Thermal Loads

3. Minimum Factors of Safety and Allowable Stresses

The factors of safety applied to the usual and unusual load cases were 3.0 and 2.0, respectively. The computed allowable stresses were then used to compare with the stresses resulting from the finite element analysis.

4. Material Properties

The concrete and rock material properties used in the finite element analysis of the dam are summarized as follows:

Property	Concrete		Foundation Rock	
Modulus of Elasticity, kPa (psi)	2.1×10^7	(3.0×10^6)	-----	-----
Modulus of Deformation, kPa (psi)	-----	-----	1.6×10^6	(2.3×10^6)
Poisson's Ratio	0.20	(0.20)	0.20	(0.20)
Compressive Strength, f'_c, kPa (psi)	28,406	(4,120)	88,804	(12,880)
Tensile Strength, f_t, kPa (psi)	4,068	(590)	-----	-----
Thermal Coefficient, m/m/°C (in/in/°F)	9.0×10^{-6}	(5.0×10^{-6})	9.0×10^{-6}	(5.0×10^{-6})
Unit Weight, kg/m^3 (pcf)	2,371	(148)	-----	-----

Method of Analysis

1. Basic Assumptions

In analyzing Cabinet Gorge Dam, it was assumed that the concrete in the arch dam and thrust blocks, as well as the rock in the foundation are homogeneous, isotropic, and linearly elastic materials. The maximum allowable tension along the base of the dam, before the concrete/rock interface was assumed to separate, was taken as 0 kPa (0 psi). The contact between the arch and thrust blocks was assumed to transmit compressive and shear loads but not tensile loads. Uplift was assumed to act on the entire foundation interface. The individual concrete blocks in the arch dam were assumed to act independently until the time they were grouted, transmitting only gravity loads until that time. The foundation rock was assumed to be weightless.

2. Computer Code

The computer code used in the analysis of the dam was ANSYS 5.0. ANSYS is a widely used, general purpose, finite element computer program developed by Swanson Analysis Systems, Inc. for structural analysis.

3. Structural Model

The structural model of the arch dam is shown in Figure 2. The finite element model of the dam was constructed of solid, shell, and contact surface elements of the ANSYS element family. The arch dam, thrust blocks, and foundation were modeled using solid brick elements. The gates and piers were modeled using shell elements. Contact surface elements

were used to model the interface between the arch dam and foundation, the thrust blocks and foundation, and the arch dam and thrust blocks. The contact surface element is able to model the no tension condition at the interfaces as well as rigid or elastic Coulomb friction.

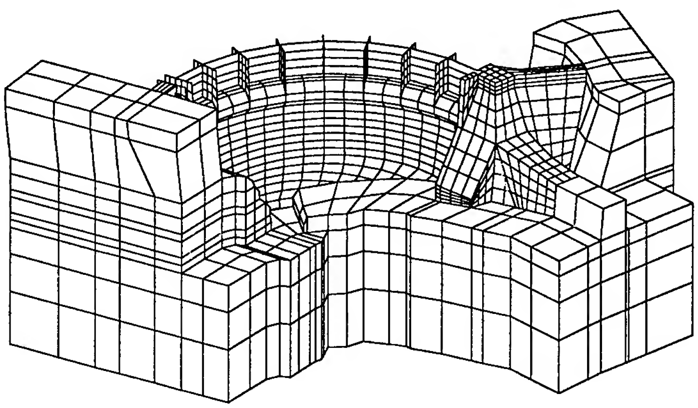


Figure 2. Finite Element Model of Cabinet Gorge Dam

Discussion of Results

1. General

The analysis results for the four load cases, are discussed in terms of computed stresses, dam deformations, and thrust block stability. The stress analysis results for the arch dam portion of the model focused on internal cantilever and arch stresses. The evaluation of the results was based on the comparison of computed stresses versus the allowable values. Where higher localized stresses were encountered, the significance of the higher stresses to the overall structural integrity of the dam was discussed. Radial displacements computed at the top of each spillway pier were tabulated and compared to actual recorded dam displacement data.

The gravity portions of the model consisted of the left and right thrust blocks and wing dam. The left thrust block and wing dam were evaluated for both internal stresses as well as for sliding along the concrete/rock interface. The right thrust block was evaluated for foundation contact stresses only. Internal stresses in the left thrust block as well as factors of safety for sliding of the left thrust block were compared to the allowable values for each respective load case considered.

2. Analysis Results

a. Arch Dam

The results of the stress and displacement analysis of the arch dam for all four load cases are presented below as maximum arch and cantilever compressive and tensile stresses.

Predominant Maximum Compressive Stresses in Arch Dam

Load Case	Arch Stress (kPa)	Arch Stress (psi)	Cantilever Stress (kPa)	Cantilever Stress (psi)	Allowable Stress (kPa)	Allowable Stress (psi)
1	3,751	(544)	8,439	(1,224)	9,467	(1,373)
2	3,227	(468)	7,488	(1,086)	9,467	(1,373)
3	5,378	(780)	8,281	(1,201)	9,467	(1,373)
4	3,509	(509)	7,977	(1,157)	14,203	(2,060)

Predominant Maximum Tensile Stresses in Arch Dam

Load Case	Arch Stress (kPa)	Arch Stress (psi)	Cantilever Stress (kPa)	Cantilever Stress (psi)	Allowable Stress (kPa)	Allowable Stress (psi)
1	179	(26)	1,379*	(200)	1,358	(197)
2	1,034	(150)	1,379*	(200)	1,358	(197)
3	510	(74)	1,379*	(200)	1,358	(197)
4	172	(25)	1,834	(266)	2,034	(295)

* Localized tensile stresses were on the order of 1,723 to 2,413 kPa (250 to 350 psi).

Maximum Computed Displacements at Crest of Arch Dam

Load Case	Description	Radial Displacement (cm)	Radial Displacement (inches)
1	Normal plus Spring/Fall Thermal	-2.84	(-1.12)
2	Normal plus Winter Thermal	-3.78	(-1.49)
3	Normal plus Summer Thermal	-1.98	(-0.78)
4	PMF plus Spring/Fall Thermal	-2.54	(-1.00)

The tabulated results demonstrate that for the most part tensile and compressive stresses for all load cases are equal to or less than the allowables. The predominant cantilever tensions shown for Load Cases 1 through 3 were found to be approximately at the allowable of 1,358 kPa (197 psi). However, the model showed some localized tensile overstressing at the toe of the arch dam near the crown cantilever, where it abutted the rock. This localized tensile effect is believed to be the result of the arch dam wanting to wedge itself into the rock foundation material at the toe. The tensile overstressed area covers approximately one hundredth of one percent of the arch dam volume and was therefore not considered

detrimental to the structural integrity of the dam. For the most part, tensile stresses were observed within the body of the dam and not at the dam/foundation contact.

The computed radial displacements in the model showed the winter to summer seasonal differential displacement to be on the order of 1.8 cm (0.71 inches). The measured winter to summer seasonal differential displacements in the arch dam are on the order of 2.0 cm (0.80 inches), which agrees very well with the analysis results.

b. Thrust Block/Wing Dam

The maximum computed tensile and compressive concrete stresses for the left abutment thrust block/wing dam section are as follows:

Maximum Tensile and Compressive Stresses in Thrust Block/Wing Dam

Load Case	Compressive Stress		Allowable		Tensile Stress		Allowable	
	(kPa)	(psi)	(kPa)	(psi)	(kPa)	(psi)	(kPa)	(psi)
1	3,951	(573)	9,467	(1,373)	848	(123)	1,358	(197)
2	5,247	(761)	9,467	(1,373)	958	(139)	1,358	(197)
3	3,861	(560)	9,467	(1,373)	972	(141)	1,358	(197)
4	4,192	(608)	14,203	(2,060)	565	(82)	2,034	(295)

The results demonstrate that the stresses are less than the allowables for all the load cases considered.

The results of the concrete/rock interface sliding analysis of the left abutment thrust block/wing dam section are as follows:

Sliding Factors of Safety for Thrust Block/Wing Dam

Load Case	Normal Force		Shear Force		FS	Minimum "C"	
	kN	(kips, x 10^5)	kN	(kips, x 10^4)	(ϕ=50°)	kPa	(psi)*
1	516	(1.159)	330	(7.411)	1.86	393	(57)
2	542	(1.218)	345	(7.750)	1.87	407	(59)
3	500	(1.124)	306	(6.887)	1.94	338	(49)
4	437	(0.982)	332	(7.470)	1.57	152	(22)

* Minimum "C" required to produce FS=3.0 for Load Cases 1, 2, and 3 and FS=2.0 for Load Case 4.

As can be seen from these results, if a base friction angle of 50 degrees (which includes the effects of foundation asperities) is used and cohesion is neglected, the computed factors of safety for the normal load cases fall below the FERC's minimum required factor of safety of 3.0. Similarly, for the

PMF load case, a factor of safety of 1.57 is computed if cohesion is neglected. In order to produce safety factors that achieve the FERC's minimum required values, a base cohesion value of 407 kPa (59 psi) is required. The analysis results presented above neglect the frictional resistance between the west face of the thrust block and the abutment rock. If the resistance provided by that face were included in the sliding analysis, the FERC's required factors of safety would be achieved without cohesion, as is shown below:

Sliding Factors of Safety for Thrust Block/Wing Dam
(including resistance provided by west face)

Load Case	FS (ϕ=50°)
1	3.41
2	3.21
3	3.87
4	3.17

Conclusions

From the stress and stability analyses of the arch dam and left abutment thrust block, we were able to demonstrate that the dam was able to withstand all of the applied loads and load cases considered. The positive results of the finite element analysis presented herein was largely due to the analysis techniques as well as the analysis tools employed in the finite element analysis. Using ANSYS Revision 5.0, with all of the available features such as the non-linear contact surface element and the birth and death capability made it possible to realistically model the dam and the adjacent thrust blocks. Using the contact surface elements, it was possible to model the concrete foundation interfaces as well as concrete to concrete joint interfaces (where tension capability does not exist), and being able to include the effect of frictional resistance where applicable. Using the birth and death capability, it was possible to simulate the actual construction sequencing of the arch dam section including the effect of joint grouting. Using the color contour plot capability to display the various output items made it very easy to evaluate the output data.

With the currently available computing capability and software sophistication in the area of non-linearities, it is now possible to model many items which previously could only be discussed when using linear finite element analysis programs. Using non-linear analysis techniques could ultimately decrease the cost of such finite element analyses in that there may be less need for follow-up work in order to explain away high contact tensile stresses as well as other modeling anomalies.

An Experimental Investigation on The Cavitation Pressure Pulsations in The Draft Tube of A Turbine

Pei zhe yi[1] Li shen cai[2]
Chen de xin[3] Bai jia cong[4] Wang cheng xi[5]

Abstract

This paper presents a preliminary research result about the characteristics of the draft tube pressure pulsations (DTPP) of the dynamic cavitation flow. The DTPP are classified and the relations are studied between the characteristic frequency and the turbine operating conditions in some typical cavitation patterns based on the statistical analysis.

Introduction

The DTPP is one of the main problems of a turbine in operation. It is the high complexity of the whirl motions and pressure pulsations as well as the limitation of the computation tool and the measuring means that the studies on the DTPP were often limited to the whirl frequency and characteristics of the vortex band; As the development of the science and technology, especially the progress of the computer and the improving of the high speed real time computer date acquisition system and the miniature pressure transducer with the high frequency response. The study on the characteristics of the real dynamic cavitation flow pressure pulsations in the draft tube can be carried on from the point of the statistical view.

Experimentation

The experiment was performed on the closed cavitation experiment stand. Figure 1 is the diagram of the experiment stand in the next page. The model

1. Engineer, Power Dispathing and Communication Bureau, Ministry of Energy, 100761, China.
2. Professor, North China Institute of Water Conservancy and Hydropower, 056021, China.
3. Vice-professor, North China Institute of Water Conservancy and Hydropower, 056021, China.
4. Vice-professor, North China Institute of Water Conservancy and Hydropower, 056021, China.
5. Vice-professor, North China Institute of Water Conservancy and Hydropower, 056021, China.

turbine was HL160-25 with the transparent throat and a transparant window in the draft tube elbow, through which the cavition phenomena can be visualized.

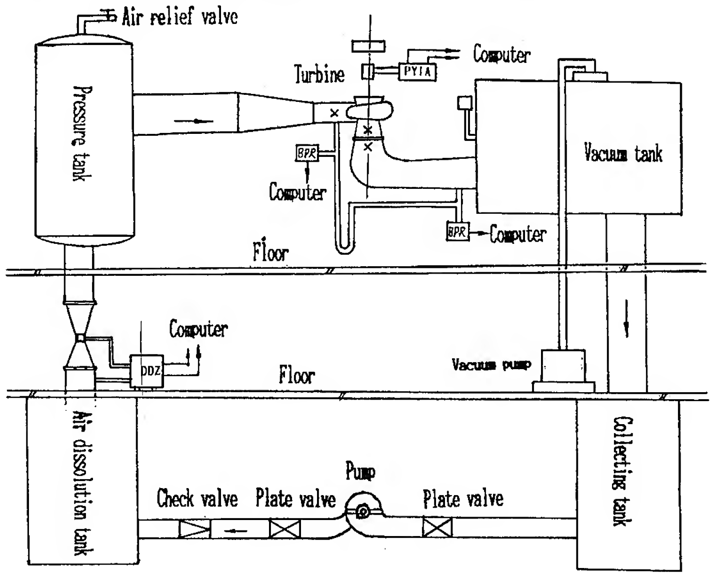


Fig. 1. Closed cavitation experiment stand

Five minature pressure transducers were used in the experiment, of which four were symmetrically installed in the draft tube elbow, one in the penstock exit to the scroll case (see figure 1, X represents the installation place of the pressure transducers).

Thirteen operating points were selected along the iso-opening lines (a_0) for the experiment, so as to visualize and analyse the DTPP in the different operating conditions, which are list in the table 1.

table 1. the parameters of the operating points for the experiment

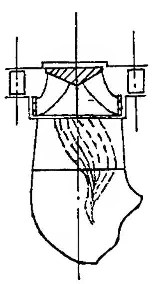
a_0(mm)	10	14	18	20
n_1'(r/min)	0, 45.93 66.34	56.13, 66.34 76.54	81.65, 51.03 61.24, 71.44	51.03, 61.24 71.44, 81.65

The signals of the DTPP together with the parameters of the operating conditions in the experiment were acquired and recorded for analysis with the help of the high speed real time computer data acquisition system.

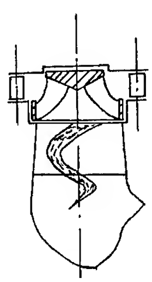
Experiment results

I. Experiment observations

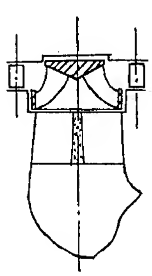
Five type of cavitation patterns were observed in the experiment under the different operating conditions and the different Thoma Numbers. They are conical cavity vortex band, spiral cavitation vortex band, straigth certral cavitation vortex band, steam-liquid two-phase flow and non-cavitation flow. Different cavitation patterns are showed in figure 2 with the exception of non-cavitation flow.



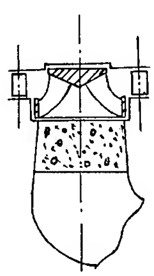
1.Conical cavitation vortex band



2. Spiral cavitation vortex band



3.Straight certral cavitation vortex band



4.Steam-liquid two-phase flow

Fig. 2. Cavitation patterns diagram

I. Date analysis

With the FFT and Auto-power spectrum, the DTPP are classified based on the statistical analysis under the different operating conditions.

1. From the point of frequeny distribution view, the dominant frequencies can be relatively classified as following :

a. Lower-frequency pressure pulsations. Frequency range: 1.22~10 HZ.

b. Mid-frequency pressure pulsations. Frequency range: 50~90 HZ.

C. High-frequency pressure pulsations. Frequency range: 150 HZ or more.

2. From the patterns of the Auto-power spectrum or according to the combined form of the dominant frequencies , the DTPP can be classified as following :

(1). Single peak order state. There is only one dominant frequency in the auto- spectrum (see figure 3), which appears in three forms asfollowing :

a. Only lower-frequency component involved. The relation between the frequency (f_{11})and the speed frequency (f_n)of the turbine is :

$$f_{11} = f_n \, / \, (2.2 \sim 4.8) \text{ HZ}$$

b. Only high-frequency component involved . This case appears seldom.

c. Olny mid-frequency component involved . This DTPP have several characteristics as following :

- —— It appears almost under every operating condition with a special Thoma Number.
- —— It appears almost under the critical cavitation conditions.
- —— It is of high penetrability which can cause pressure pulsation with the same frequency in the penstock exit to the scroll case , or partial system resonance in another words (see figure 4). This is so called the effect of "cavitation resonance" (Li ,Zhang and Hammitt, 1986).

The statistical relations (see figure 5) between the frequency (f_{1m}) and the critical Thoma Number (σ_{pc}) and the parameters of the operating conditions can be expressed as:

$$f_{1m} = 0.985n_1'+7.68 \text{ (HZ)}$$

$$\sigma_{pc} = -0.0145A0+1.47$$

where n_1' - unit speed (r/min), A0 - guide vane opening (mm)

(2). Double peaks order state. There are two dominant frequencies in the Auto- spectrum (see figure 3). It appears in following constitutional forms:

a. Lower-frequency (f_{21}) and high frequency(f_{2h}) ,and

$$f_{21} = f_n/(2.2 \sim 4.8) \text{ (HZ)}$$

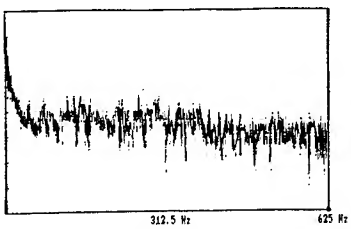
$$f_{2h} = 2.62n_1'+55.1 \text{ (HZ)}$$

where: f_n, n_1' represent the same meaning as mentioned above.

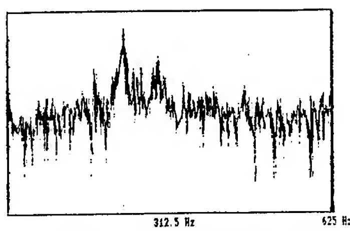
b. Mid-frequency and high frequency, which appears seldom .

c. Mid-frequency resonance. There are a mid-frequency component and its quadratic resonance frequency component in the Auto-spectrum , which appears also seldom .

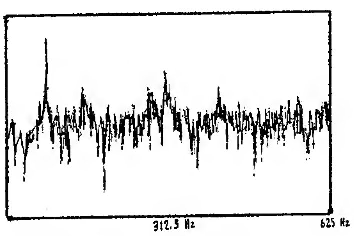
(3). Three peaks order state . The lower-frequency component (f_{31}) , mid- frequency component (f_{3m}) and high frequency component (f_{3h}) take the equivalent place in the Auto-spectrum, which is a typical DTPP (see figure3). The statistical relations (see figure 6) between the frequencies and the operating conditions can be expressed as following :



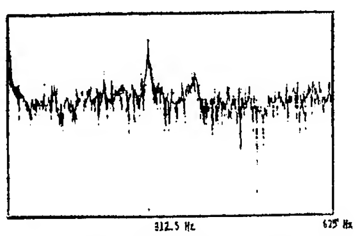
1.Single peak order state (a)



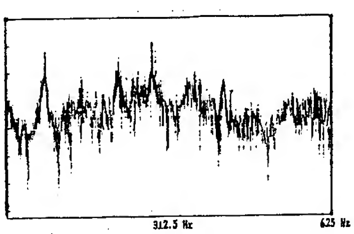
2.Single peak order state (b)



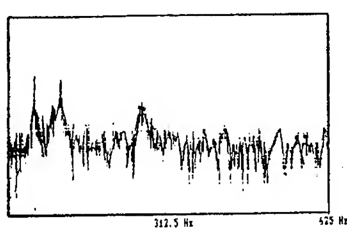
3.Single peak order state (c)



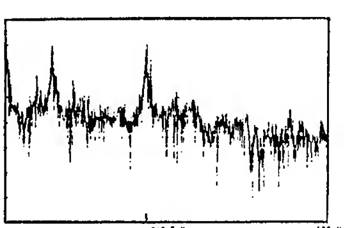
4.Double peaks order state (a)



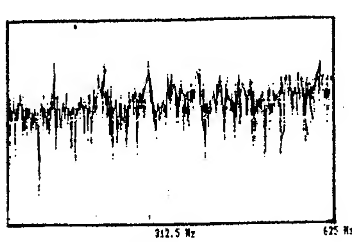
5.Double peaks order state (b)



6.Double peaks order state (c)



7.Three peaks order state

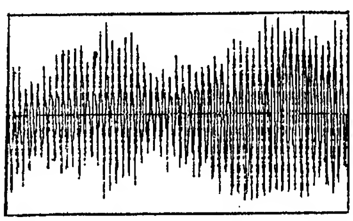


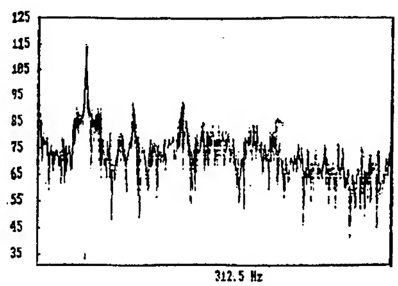
8.Chaos state

Fig. 3. The patterns of the Auto-power spectrum

This is the wave of time function

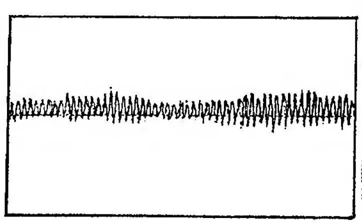
This is the Log Power Spectrum

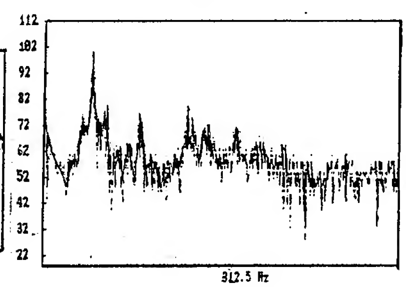




a1

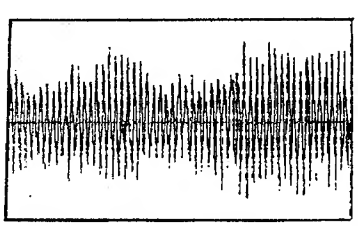
a2

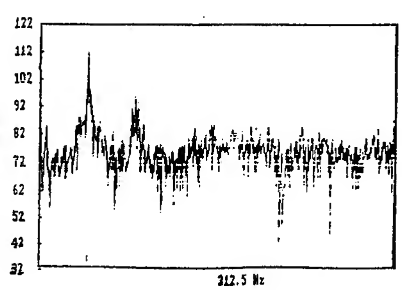




b2

b2





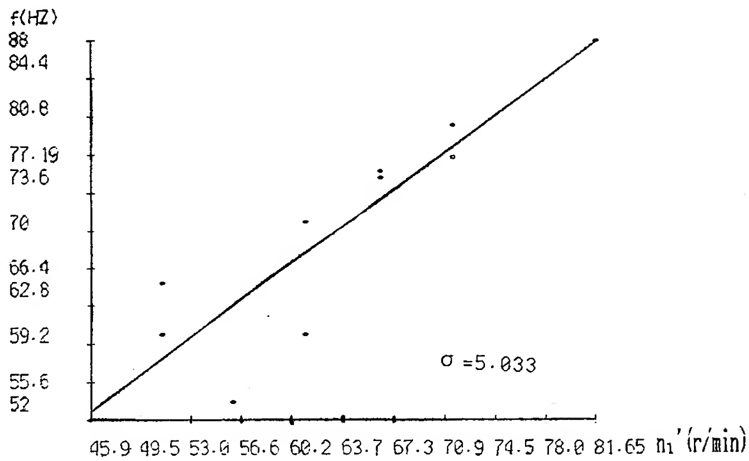
c2

c2

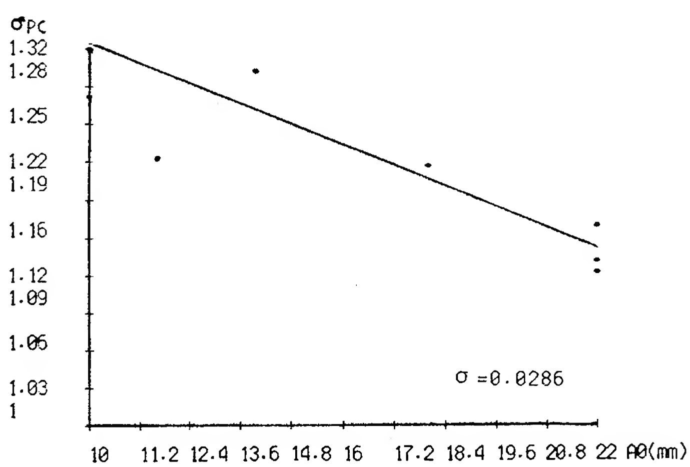
a1, a2 and b1, b2 represent the wave of time function and the Log Power Spectrum of two pressure transducers installed in the draft tube elbow.

c1, c2 represent the wave of time function and the Log Power Spectrum of the pressure transducer installed in the penstock exit to the scroll case (see figure 1).

Fig. 4. The single peak order state only with the mid-frequency component involved

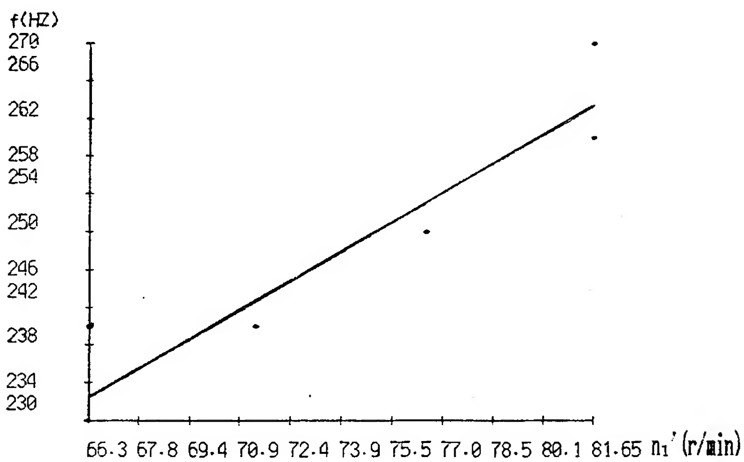


1. Regression line of the mid-frequency component and the unit speed

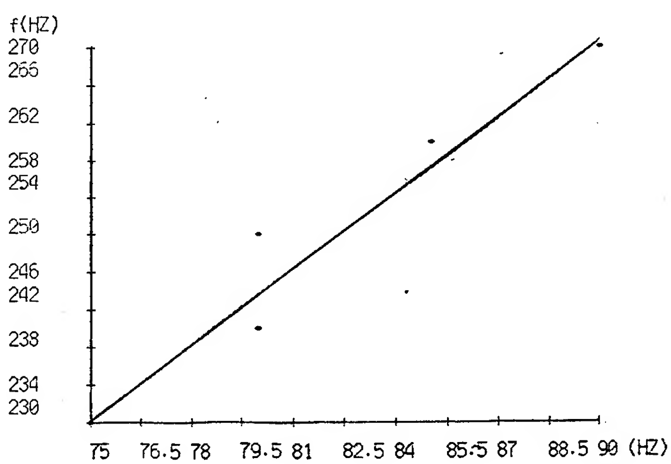


2. Regression line of the critical Thoma Number and the guide vane opening

Fig. 5. The statistical characteristics of the single peak order state only with the mid-frequency component involved



1. The regression line of the high-frequency and the unit speed



2. The regression line of the high-frequency and the mid-frequency

Fig. 6. The statistical characteristics of the three peaks order state

$$f_{31} = f_n/(2.2\sim4.8)\ (HZ)$$
$$f_{3h} = 2.03n_1'+97.51$$
$$= 2.7f_{3m}+27.41$$
$$\approx 3f_{3m} \quad (HZ)$$

At the same time , the evident cavitation vortex band can be clearly visualized. According to the analysis mentioned above ,We can infer that:

f_{31} is the frequency of the vortex band whirl.

f_{3m} is the frequency associated with cavitation.

f_{3h} is the frequency of the f_{3m}' cubic resonance.

(4). Chaos state. There is not any dominant frequency in the Auto-spectrum. (see figure 3)

Concluding remarks

1. Preliminary classification and analysis of the DTPP were carried out in this paper. The further study is however reqired on the points of views because of the limitation of the time and the experiment conditions.

2. The occurence of the initial cavitation can be predicted in the light of the characteristics of the DTPP under the critical cavitation condition which is the effect of the "cavitation resonance" with the frequency of 50~90 HZ in this experiment.

3. The cavitation state in the draft tube can be inferred in the light of the research results, So it can be supervised in line according to the frequency analysis of the DTPP.

4. The research results would provide the new material for the study on the vibration of turbines.

Appendix 1, References

Shengcai Li, Youjing Zhang and F. G. Hammitt, (1986) , "Characteristics of cavitation bubble collapse pulses, associated pressure fluctuation, and flow nose". Journal of Hydraulic Research, VOL. 24. 1986. NO. 2.

Conceptual Design and Physical Model Testing of the White River Diversion Dam

Al Babb[1], Michael Blanchette[2], and Robert King[3]

Abstract

The 82-year-old White River Diversion Dam and Intake need replacement and modification. The dam experiences an annual sediment load averaging 500,000 tons as well as large floating debris. Through sequenced engineering and physical modeling efforts a unique design has been developed to address these challenges. The new design uses structural walls, radial gates, rubber weirs and intake structure modifications to work with the river to address these challenges.

Introduction

Puget Sound Power & Light Company's White River Diversion Dam was constructed in 1911 at Buckley, Washington. The dam's primary purpose is to divert up to 2,000 cfs, the Project's water right, into Lake Tapps for power generation at the 63.4-MW White River Powerhouse. The existing dam and intake have been the subject of a recent engineering design and physical model study as part of Puget Power's licensing plan for the project. Construction is scheduled to begin on issuance of a license from the Federal Energy Regulatory Commission.

The White River is glacially fed from the northern slopes of 14,410-ft high Mt. Rainier. About 500,000 tons of sediment arrives at the diversion dam in an average year. The focus of this paper is on the development of a dam and intake design capable of excluding bedload from the intake and passing it downstream of the dam. The design, which was developed by a sequenced engineering and physical model study, produced a unique physical structure that uses the natural hydraulic river forces to provide the desired capabilities of the diversion works.

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[1] Principal, Northwest Hydraulics Consultants, Vancouver, B.C.
[2] Senior Engineer, Project Mgmt., Puget Sound Power & Light Co., Bellevue, WA
[3] Hydropower Section Manager, HDR Engineering, Inc., Bellevue, WA

Background

The existing diversion dam is 352 ft long and 11 ft high (Figure 1). It is composed of a 4-ft high combination rock and concrete filled timber crib dam with 7-ft high flashboards. The existing intake, located immediately upstream of the diversion dam, currently ingests significant quantities of sediment in an average year, 100,000 to 200,000 tons. To avoid ingesting large quantities of bedload during high flow events, diversion is curtailed. This prevents the occasional blockage of the intake from bedload deposition. Debris jamming of the intake can also occur during high flows, creating a loss of diversion capability.

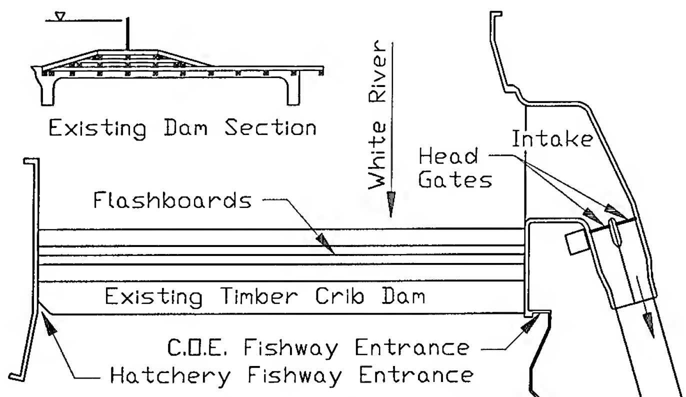

Figure 1 - Existing Dam & Intake

Because the existing intake invert is set at elevation 662 fmsl, sediment is readily diverted into the project flowline along with diverted water. To maintain full diversion during the rising leg of the flood hydrograph, Puget Power operates seven removable steel panels (6-ft by 7-ft high each) located on the left side of the dam to pass excess flows. When the steel panels are open, some of the bedload arriving at the dam passes through the panel openings. The flashboards were designed to fail under higher flow conditions thereby allowing the dam to pass large flood flows. Large volumes of sediment are also passed downstream of the dam along with the flood flows. Upon flashboard failure, the diversion capacity of the intake is reduced severely until the flashboards are reset, sometimes taking days or weeks to restore. However, even with flashboard failure, the intake continues to ingest large volumes of sediment.

The need to ensure the operation of two existing fishways during proposed construction activities and to enhance their long-term operation has been considered throughout the design process. The left bank fishway serves the U.S. Army Corps of Engineers Buckley Trap and Haul Operation. The right bank fishway serves the White River Hatchery located adjacent to the dam.

Design

The proposed new construction serves six primary functions. Replace the aging timber structure; improve the ability to exclude bedload from the intake; effectively pass excluded bedload downstream of the dam; prevent debris from entering the intake; provide debris handling features, and improve the intake hydraulics to provide uniform diversion of the 2,000 cfs water right.

Dam/Intake Alignment -- The diversion dam is located at about the midpoint of a sweeping 1,500-ft radius, 150 degree bend in the river. The next bend upstream is a sharp "dog leg" armored with 18-inch rock to prevent bank failure and overtopping, which could threaten the hatchery. A review of aerial photographs dating back over 50 years shows that the river has stabilized in its present course and that major changes in river geometry within this reach are highly unlikely due to the flood flow regulation provided by Mud Mountain Dam.

The main channel of the river arriving at the dam site is about 150 ft wide and is located on the left bank, the outside of the bend. Immediately upstream of the dam the river has cut a 30-ft high vertical bank that is slowly retreating at a rate of about 1 in/yr. The main channel runs along the toe of this cut bank immediately before reaching the intake. This geometry produces a river thalweg positioned along the left bank. The helicoidal flow associated with this bend tends to move sediment from the outside of the bend along the channel floor toward the inside of the bend and has produced a point bar on the inside of the bend.

The positioning of the proposed new dam and intake modifications required careful study of existing and future physical structures and hydraulic conditions for the site. Based on the combined efforts of an engineering and physical model study, the proposed design was developed (Figure 2).

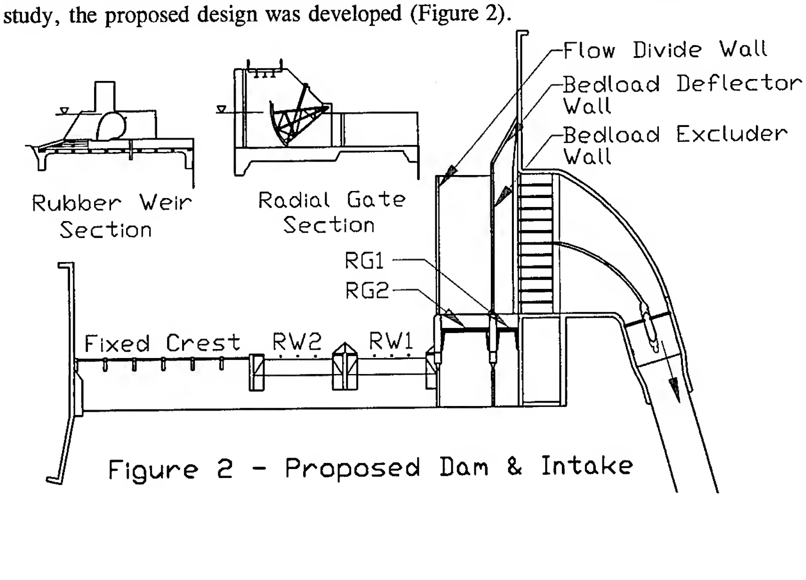

Figure 2 - Proposed Dam & Intake

The existing face of the intake is positioned about 30 ft back from the main channel and produces a hydraulic dead zone between the edge of the main channel and the intake face.

The proposed intake design moves the intake face 30 ft out toward the main channel while retaining an orientation tangent to the river's main channel. The proposed design also provides a submerged headwall running the full length of the 100-ft long, 12-bay intake structure. This positioning, alignment and structure are intended to take advantage of the river's hydraulic sweeping capabilities to move bedload and floating debris downstream of the intake and past the dam. The relocation of the intake face and additional structural changes to the intake are also valuable in providing uniform diversion capabilities of the full 2,000 cfs under a wide range of flows while excluding and passing bedload as discussed in greater detail below.

Relocation of the existing dam axis alignment upstream or downstream was considered during design. However, the axis location is constrained by the presence of the intake upstream and the two fishways downstream. Positioning of the proposed new dam gates upstream and downstream of the axis has been accomplished. The two gates located on the left side of the dam (radial gates) have been positioned close to the intake to facilitate the hydraulic action of the gates to move bedload and debris past the dam. The other two gates (rubber weirs) have been positioned to allow reuse of the existing timber crib dam foundation, expedite construction, and allow for vehicular access to the downstream side of the structure. A fixed crest overflow section comprising the balance of the replacement dam is located along the same axis as the rubber weirs for the same reasons.

The horizontal positioning of individual gates along the dam axis has also been considered during design. Under existing river conditions, the main channel of the river is located along the left bank, both upstream and downstream of the dam. The gate positioning along the dam axis associated with the proposed replacement dam has been developed to keep the main channel of the river located along the left bank. The relatively deep inverts, 657 fmsl, of the two radial gates, located on the far left of the proposed replacement dam, keep the thalweg and main channel of the river located along the left bank under high flow conditions when the river is most powerful and capable of altering its alignment.

The point bar mentioned above exists both upstream and downstream of the dam, along the inside of the bend. Accumulation of sediment behind the flashboards upstream of the right half of the dam occurs over time and eventually reaches the elevation of the top of the flashboards. The failure of the flashboards under high flow conditions results in the temporary scouring of deposited sediments in the local area upstream of flashboard failure. Resetting of the flashboards stops the scouring action and begins the deposition pattern again. The proposed design does not require gating the full 352-ft width of the existing dam crest to pass the 18,400 cfs inflow design flood (IDF). A fixed crest section of dam will replace the

flashboards along the right 127$\pm$ ft of dam structure. Under extreme flood flow conditions the rubber weirs are deflated to pass the IDF. The rubber weirs will operate much like the existing flashboards do under high flow conditions.

Gate Selection -- The need to pass large volumes of bedload coupled with the need to maintain headwater elevation control to ensure proper intake operation, resulted in a determination that an undershot-type gate was required as the primary dam regulating gates under normal flow conditions up to moderate flood flows. This determination was further refined to select radial gates as the under shot gate, based on the severe abrasive operating conditions the gates would experience because of the volume of sediment that will pass through the gates. The sediments arriving at the dam range in size from fine silts and sand to gravels and well rounded cobbles up to 3 ft in diameter. Approximately 75% of the bedload is smaller than 2.5-inches, and the balance is larger material. The gate lip of the radial gates will be replaceable, and both the lip and the imbedded gate side seal plates will be made from abrasion resistant metals.

The 35-ft wide radial gate (RG2) can pass about 4,500 cfs while maintaining headwater control and full intake diversion. Inflatable rubber weirs were selected to pass the next increment of flood flow from 4,500 to 12,000 cfs when the rubber weirs are fully deflated and the headwater is allowed to rise 1-ft. For flows higher than this, the 16-ft wide radial gate (RG1) is raised. Finally, both the radial gates are raised clear of the water, allowing unregulated passage of flood flows arriving from Mud Mountain Dam.

The rubber weirs (RW1 and RW2) were chosen for several reasons: economy, ability to reuse the existing dam foundation, speed of construction preparation and installation, failure to the open (deflated) position, and hydraulic compatibility with the natural river cross section as mentioned previously. The IDF can be safely passed with the gate types selected for the design.

The invert elevations for the two radial gates were set to keep the bedload low in the water column as compared to the intake floor invert. The radial gate inverts are at elevation 657 fmsl as compared to elevation 662 fmsl for the intake floor. The invert of the radial gates and approaching gate bay slabs, combined with the height of the intake invert and the flow directing walls, prevent bedload from entering the intake. The invert for the rubber weirs, elevation 663 fmsl, was set based on the required capacity needed to pass the IDF and the structural requirements for re-use of the existing dam foundation.

Flow Directing Walls -- Integral to the successful design are three specific walls. These structures create hydraulic conditions which serve three primary functions. Diverting 2,000 cfs, preventing bedload greater than 1/2-inch in diameter from entering the intake, and effectively moving bedload past the dam. The three walls are the bedload deflector, bedload excluder and flow divide walls.

Most bedload arriving at the dam is present in the main channel. As the main channel flow approaches the dam, the first wall encountered is the 7-ft high submerged bedload deflector wall. The deflector wall is an angled 38-ft long wall located immediately upstream of the intake. This wall uses the natural helicoidal rotation of the flow to deflect the bedload to the right of the deflector wall and out into the approach channel for RG2. The downstream end of the deflector wall is the upstream end of the submerged bedload excluder wall. The excluder wall is 109 ft long, varies in height from 7 to 9.4 ft, and runs the full length of the intake terminating at the pier common to the two radial gates. The excluder wall prevents the majority of deflected bedload from jumping over into the RG1 bay where it could potentially be drawn into the intake. Bedload that makes it over the deflector or excluder walls settles into the approach channel to RG1. In combination, these walls serve as the primary defense for preventing bedload from entering the intake.

The flow divide wall is 100 ft long and separates RG2 from RW1. The wall varies from 15 to 17 ft in height and the top of the wall is always above the water surface. The flow divide wall performs two important hydraulic functions. First, it allows the radial gates to maintain a relatively constant headwater elevation in front of the intake even with the rubber weirs in a deflated position. Second, in combination with the excluder wall, it creates a relatively narrow confined channel approaching RG2. Velocities created by this confined channel are sufficient to move bedload along the paved sloping apron and beneath the radial gate. The wall initially created a hydraulic restriction to flow into the intake, which required considerable physical modeling to resolve.

The aprons approaching the two radial gates are concrete paved and slope downward to the radial gates. They promote the easy movement of rolling bedload past the intake and beneath the radial gates. The discharge aprons below the radial gates are also concrete paved to prevent scour and to promote bedload movement past the dam. Armoring of the gate bay floors and a short distance up the gate bay side walls will provide protection of the concrete structure from abrasion. A submerged roller bucket downstream of RG2 enhances hydraulic conditions at the left bank fishway entrance while reducing downstream scour.

Under low flows, bedload will accumulate in the approach apron to RG2 until the gate is operated, at which time the approach channel is evacuated. Under high flows, bedload is swept directly under the radial gates and passed downstream of the dam since the radial gate is in the open position. Occasional accumulation of bedload in the radial gate approach channels under high flows can be cleared by raising the radial gates free of the water. This produces high velocities that sweep the bedload out of the channels and downstream of the dam. Operation (deflation) of the rubber weirs under flood conditions evacuates bedload accumulated in the zone immediately upstream of the rubber weirs. Additionally, bedload is carried with the flow passed over the fully deflated rubber weirs and past the dam.

Intake Modifications -- The existing intake is 80 ft wide with steel beams for stop log slots and the gross average velocity is 2.8 fps. Flow distribution across the intake face is poor and dewatering is extremely difficult due to sediment accumulation. Regulation of diverted flows is provided by two gear-driven 14-ft wide vertical slide gates which will be refurbished. Specific improvements were made to the intake to exclude bedload; prevent the entry of large floating debris; improve hydraulic uniformity; provide dewatering capabilities and ease of maintenance; and allow for the potential future addition of trashracks.

A headwall having a submergence of 36 inches runs the full length of the intake. Floating debris is directed along the face of the intake along the headwall. Debris that becomes lodged in front of or in the intake will be removed by a mechanical debris handler located on both the intake and radial gate operating decks. The proposed new intake is 100 ft wide and is divided into 12 equal 7.5-ft wide bays, each equipped with stop log slots. A new intake divider wall, will be located between intake bays 6 and 7 and run to the pier common to the two slide gates. The new divider wall coupled with the 12 bays, results in nearly uniform flow distribution across the intake face. An intake floor extension slab located on the intake face will allow for the future addition of a trash rack if desirable.

Physical Model Study

The model, constructed on an undistorted scale of 1:40, reproduced 1,500 ft upstream from the dam, and 450 ft downstream. The study was conducted in accordance with the Froude criterion. Incoming flow to the model was measured with orifice meters. Weirs were used to measure flow exiting the model.

Bedload -- The model bedload material was selected to reproduce observed prototype movement, consisting of vigorous overall rolling motion at 8000 cfs, and some movement of the finer components down to about 4000 cfs. A mixture of sands was developed by pretesting in a flume before placement in the model. The model material ranged in diameter from 0.3 mm (0.012 in) to 4.75 mm (0.19 in), and had a mean of 1.2 mm (0.047 in). At the 1:40 scale, the prototype range is 12 mm (0.47 in) to 190 mm (9.5 in), and the mean diameter 48 mm (1.9 in). The deposition of finer gravels and sand was qualitatively observed with the use of finer sand and crushed walnut shell (specific gravity 1.3). Although the incoming bedload transport rate was not simulated, and no estimates were made for deposition times, sediment evacuation rates were estimated. The sediment feed rates used to study bedload passage are believed to exceed prototype rates, suggesting that model-based predictions are conservative.

Baseline Tests -- Tests were conducted with the existing structure to confirm that the model capably reproduced existing river characteristics, and to establish baseline conditions that could be used for comparison with the proposed design. Bedload movement in the model initiated at about 4,000 cfs, and was fully developed at 8,000 cfs, corresponding to the desired degree of bed mobility. The

model reproduced the scoured leftside channel from the bend immediately upstream of the dam down to the dam. The model also reproduced the movement of material to the inside of the bend, the rightside point bar buildup, thalweg location and accurate simulation of upstream and downstream flow patterns. Diversion flow turning into the intake with no spill produced a large eddy in the left side of the intake, resulting in deposition of both fine and coarse sediments. Headloss through the intake for 2,076 cfs diversion flow with no spill, and with deposition in the intake, was 0.92 ft. Most of the loss occurred as flow turned into the intake from the river. For a diversion flow of 2,000 cfs and spill of 5,500 cfs through the steel gates, considerable quantities of bedload passed into the intake, while little bedload was transported through the open steel gates.

During the rising leg of the hydrograph, downstream scour for a 5,500 cfs spill progressed to elevation 641 fmsl, 19 ft below the existing apron. Maximum scour remained at this elevation for spill flows up to 10,000 cfs, then aggraded slightly to elevation 644 fmsl as the flow was increased to 13,000 cfs. Prototype scour is known to have progressed to at least elevation 644 fmsl, the deepest level of recently deposited alluvium reached in a track-hoe test pit. The baseline testing confirmed that the physical model accurately simulated actual conditions quite well.

Bedload Passage -- During initial tests the RG2 channel was sloped at 4% with its upstream end at elevation 661 fmsl. The shallow depths at the upstream end created a hydraulic restriction for diversion capacity. For this arrangement, bedload passage was most effective when all spill was passed to the right of the divide wall, effectively carrying the bedload with the flow. However, this condition created high scour-producing velocities at the upstream end of the divide wall, requiring large diameter riprap armoring. Lowering the RG2 channel slope to 2% (upstream channel invert lowered to elevation 659 fmsl) allowed spill through RG2 with only modest overtopping of bedload into the RG1 channel. Buildup of deposition in both RG1 and RG2 channels and subsequent passage of bedload through the intake was controlled with periodic evacuation. Evacuation is most effective when both RG1 and RG2 are open fully and concurrently.

Debris Passage -- Most debris in the White River collects in the Mud Mountain reservoir located 5 mi upstream from the diversion dam. Debris added in the 5 mi reach upstream of the diversion dam will be mechanically removed or hydraulically moved through gates. Most debris arriving at the dam reached the area between the intake and the flow divide wall, and very little debris passed to the right of the flow divide wall, even if flow passed over RW1. Any debris that passed to the right of the divide wall, with RW1 operating, passed over RW1. Most debris collected at RG1. The clappette in RG1 will be used to pass this debris downstream. Floating debris occasionally passed beneath the radial gates. A few of the smaller pieces passed under the submerged intake headwall and some debris became lodged at the intake face.

Diversion Flow -- Improved uniformity of flow through the intake structure was achieved two ways. Providing 12 separate intake bays with the upstream pier noses aligned with the approach bank so that they functioned as turning vanes, and providing an intake divide wall downstream from the central intake pier. Although the pier noses and the intake divide wall successfully distributed the flow among the twelve bays, flow within each bay concentrated on the right side (viewing in downstream direction). Finer sediment tended to settle in an eddy produced on the left side. For spill conditions where most or all of the spill passed over the rubber weirs to the right of the divide wall, the water surface upstream from the radial gates elevated, and flow uniformity through the intake bays greatly improved. Energy losses through the intake of 1.09 ft for the left half and 0.88 ft for the right half do not differ greatly from those measured for the existing intake.

Downstream Flow Patterns and Scour -- There are two significant features of the flow downstream from the dam. The creation of flow patterns conducive to the attraction of upstream travelling anadromous fish to the left and right bank fishways, and the control of scour along the downstream face of the structure. Initial testing was conducted with a level 47.5-ft long apron downstream from the radial gates. This apron was not deep enough to generate a hydraulic jump and flow exited the apron at relatively high velocity and at a level below tailwater elevation. Flow exiting from RG2 caused large, strong eddies to form on either side of the jet. The leftside eddy produced high velocity return currents adjacent to the left bank fishway entrance that were considered detrimental. In addition, high velocity flow along the downstream face of the structure, particularly on the right side, scoured the downstream bed to elevations as low as 637 ft, 20 ft below the apron.

To minimize the effect of the return eddy on fish attraction, initial spill during periods of fish passage will be passed through RG1 and a bucket type dissipator with a 14-ft radius will be used for RG2 to create a contained hydraulic jump. Even at the most severe scour flow of a 8000 cfs, much of the jump still retained in the bucket. The elevated water surface in the boil at the end of the bucket reduces the magnitude of the return flow velocity, improves flow patterns for fish attraction, and reduces scour depths. Velocity measurements showed the dam to be an effective barrier to upstream fish migration as desired.

Capacity -- The water surface upstream from the intake produced by the combination of radial gates and rubber weirs was at elevation 672.37 fmsl for the flow of 18,780 cfs. The water level for the IDF should not exceed elevation 672.5 fmsl. With a low head dam of this type, there is a possibility that some project features may interfere with the capacity of the gates and rubber weirs. For example, the RG2 channel sloped at 4% (upstream invert elevation 661 ft) restricted the capacity of RG2. In addition, the capacity of 70-ft rubber weirs, from preliminary tests, when operating together, was substantially reduced by the

constrictive effect of the divide wall. This flow reduction was considerably less with the 50-ft rubber weirs in the proposed design.

Operation -- The model was useful in developing operating guidelines which will be used to operate the project:

- Normal upstream pool at elevation 671.5 ft for flows up to 5000 cfs, then at elevation 672.5 ft to 18,400 cfs.
- For incoming river flows below 2,130 cfs, 130 cfs will be passed through RG1 to satisfy instream flow requirements, and the remaining flow diverted into the flowline.
- For flows between 2,130 and 6,535 cfs, including 2,000 cfs diversion, the system requires the passage of various flows through RG1 first to enhance fishway attraction conditions. The remaining flow in excess of the 2,000 cfs diversion will pass under RG2.
- For flows in excess of 6,535 cfs, and maintaining full diversion of 2,000 cfs, the project will be operated so that bedload is passed downstream with a minimum of material diverted into the flowline. This involves the passage of the excess flow above 6,535 cfs over the two rubber weirs.
- For flows above about 12,000 cfs, RG2 will be opened until fully open, followed by further opening of RG1 for flows exceeding about 16,000 cfs.
- During the passage of high flows, the radial gates will be periodically fully opened to evacuate accumulated sediment from the radial gate channels and from the riverbed upstream from the channels.
- Debris accumulating near the intake will be mechanically removed or passed by opening gates as needed.
- Periodic overtopping of the fixed crest section of the dam will be used to keep a narrow channel clean to facilitate right bank fishway operation.

Conclusions

The proposed replacement dam for the White River Project uses bedload deflector and excluder walls and a flow divide wall combined with set opening sequences for the gates and rubber weirs to provide the effective bedload passage. The replacement structure uses a 12 bay intake works at the left abutment of the dam with a release system featuring two radial gates and two rubber weirs. The structure successfully diverts flow with a minimum of bedload material entering the intake. Favorable attraction flows to the left bank fish entrance were established with the riverward relocation of the entrance and the early use of RG1 to pass flows adjacent to the fish entrance to eliminate the return eddy. A bucket type dissipator is used downstream of RG2 to control scour and to reduce the magnitude of the return flow. Most debris approaching the dam will accumulate on the left bank upstream from the radial gates, from where it will be either hydraulically passed or removed mechanically. The design of the proposed facilities was studied and improved with the use of a hydraulic model. Significant benefits of the physical model included: design refinement, gate sizing, scour prediction, bedload behavior prediction, and agency demonstration and consensus building.

The Eagle Mountain Pumped Storage Hydroelectric Project

Malcolm S. Jones, Jr. Mem ASCE[1]
and
Patrick E. Slattery[2]

Abstract

The Eagle Mountain Project is a pumped storage hydroelectric facility to be located in the eastern part of Riverside County, California, in the vicinity of the town of Eagle Mountain. The Project would use two of the existing Kaiser Eagle Mountain Mine iron ore open pit excavations as upper reservoirs, and one large open pit mine as the lower reservoir. The reservoirs can be configured to contain about 200 x 10^6 m^3 of water, which represents 240 GWh of recoverable energy which could produce more than 4,000 MW of peaking power on a weekly pumpback cycle.

Introduction

The Eagle Mountain Project is a pumped storage hydroelectric facility to be located in the eastern part of Riverside County, California, in the vicinity of the town of Eagle Mountain which is about 20 km north of Desert Center, and midway between the cities of Blythe and Indio on Interstate 10. The Project site is immediately adjacent to the Joshua Tree National Monument. The Project would use the existing Kaiser Eagle Mountain Mine iron ore open pit excavations as the reservoirs. The location of the Project is shown in Figure 1.

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1 Malcolm Jones Associates, Laguna Niguel, CA 92677
2 Power Development Consultant, Inc. Greenville, SC 29615

The Project would consist of the construction of three small dams to increase the natural water holding capacity of the three major mine excavations, and the construction of an underground powerhouse complex. The intended capacity of the powerhouse is 4,000 MW which will be produced by twelve 350-MW reversible pump/turbine generating units. Total construction cost of the Project would be about $2 billion (in 1992 dollars). It is planned that the Project will be developed in 1,000 MW phases as the market for peaking power in the southwestern United States grows.

Project Features

Major features of the 4,000-MW pumped storage Project are shown in the flowsheet, Figure 2. In the pumped storage concept, water from the lower reservoir is pumped up to higher reservoirs during the night when electrical demand is low. During the day, when demand for electricity is high, the water is passed back down through the reversible pump/turbine generators producing peaking electrical capacity for the grid.

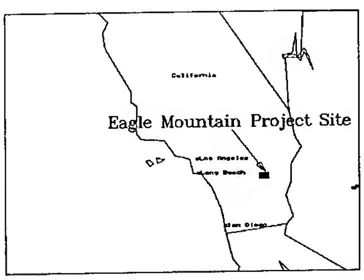

Figure 1. Project Location

For the Eagle Mountain Project, there are two upper reservoirs with dams and outlet works: the Central Deposit Upper Reservoir which is in the middle of the site with a bottom elevation of about 376 m above mean sea level (msl), and the Black Eagle Mine Upper Reservoir at the western part of the site with a bottom level of about 488 m-msl. The lower reservoir is the East Pit Mine which has a bottom elevation of about 220 m-msl. The underground powerhouse with reversible pump/turbine generator units will have waterways connecting the lower reservoir to the upper reservoirs through the powerhouse. Associated facilities include access tunnels and shafts, a switchyard, and about 230 km of power transmission facilities.

The Project is designed to have a stored volume of about 205 x 10^6 m^3 and operate with an average head of about 376 m, which will allow the delivery of 4,000 MW in a load following mode for fifteen hours, five days a week, with pumpback on a weekly cycle. The deliverable energy is about 240 GWh.

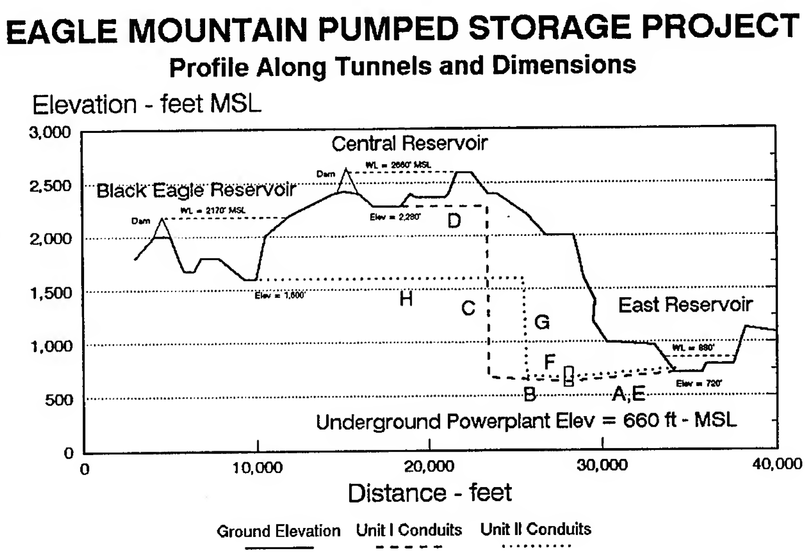

Figure 2. Project Flowsheet

Physical characteristics of the reservoirs and dams are presented in Table 1: Unit 1 identifies the facilities associated with the Central Deposit Upper Reservoir, and Unit 2 those facilities associated with the Black Eagle Upper Reservoir which is at a slightly lower elevation.

Central Deposit Upper Reservoir

The highest upper reservoir will be constructed using the Central Deposit Mine. It will have a maximum surface area of 1.26 km^2 at a maximum operating water elevation of 780 m mean sea level (msl). In addition to this maximum operating level, 3 m of surcharge will be provided for flood storage and freeboard. Maximum capacity of Central Deposit Reservoir will be 76 x 10^6 m^3 of which 75.6 x 10^6 m^3 will be available for power generation in the Unit 1 powerplant which has hydraulic equipment capable of handling heads of up to about 550 m.

The 8.4 x 10^6 m^3 of natural capacity in the Central Deposit Mine will be enhanced by construction of a partial dam of the impervious rockfill type around about one-third of the southern perimeter to raise the crest to 780 m-msl.

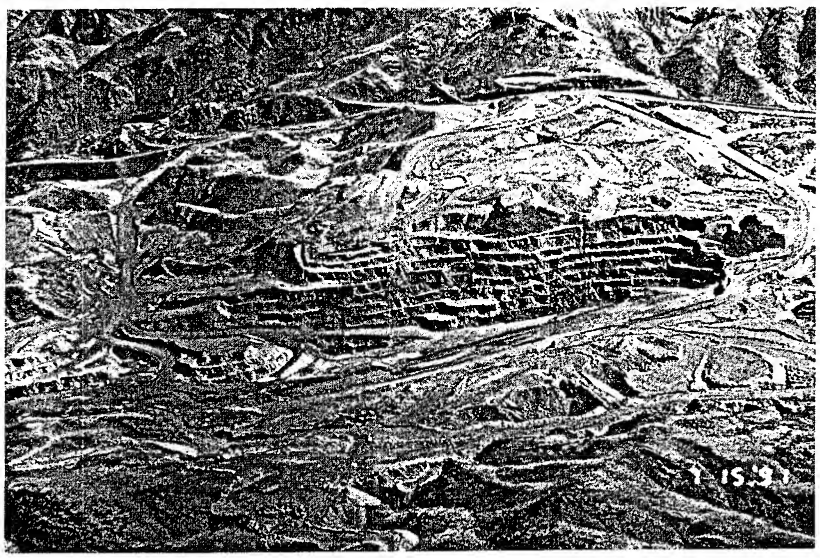

Figure 3. Photograph of Central Deposit Mine Site (Upper Reservoir Unit 1).

Black Eagle Upper Reservoir

The Black Eagle Mine, with a natural capacity of about 36 x 10^6 m^3 is shown in Figure 4. This site will be further developed by construction of an impervious core rockfill dam with a height of about 60 m at a narrow defile at the west end of the site to form the Upper Reservoir for Unit 2. With a water level of 679 m-msl, the contained volume will be about 130 x 10^6 m^3.

East Pit Lower Reservoir

The East Pit Mine will be used as the Lower Reservoir. The capacity of this mine excavation, shown in Figure 5, is about 35 x 10^6 m^3 with the natural lip at 347 m-msl. As the Project grows in output capacity, the holding capacity of the Lower rservoir can be increased by the construction of an impervious central core rockfill dam around the eastern part of the excavation to eventually raise the water level to an elevation of 475.5 m-msl with a contained volume of about 247 x 10^6 m^3. When the water level is to be raised above 381 m-msl, a central core rockfill dike will be constructed along the southern perimeter to contain that portion of the reservoir.

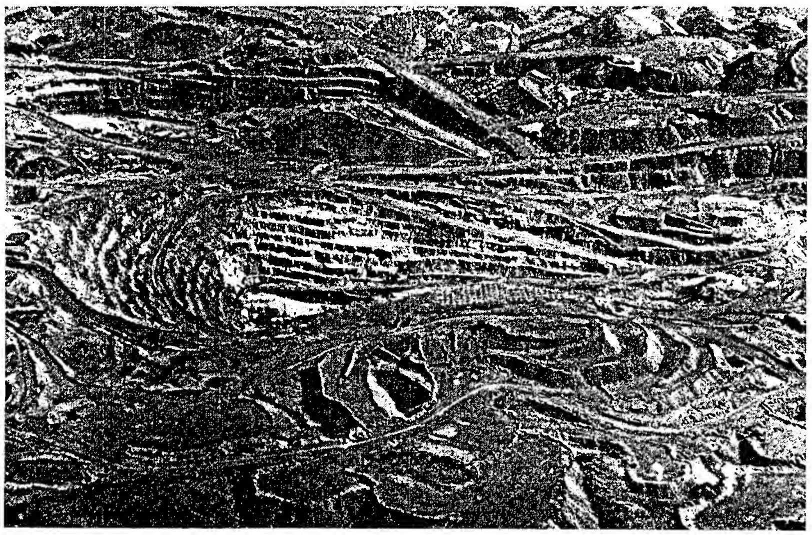

Figure 4. Photograph of Black Eagle Mine Site (Unit 2)

Figure 5. Photograph of East Pit Mine Site (Lower)

Pumped Storage Power Plant

The powerplant will be located about 400 m underground near the East Pit Reservoir. The entire complex will contain three main chambers, i.e., powerplant, transformer, and tailrace gate. The access tunnel to the complex, sized large enough to accommodate a three-phase transformer, will extend to the surface at the west side of the town of Eagle Mountain.

Powerplant Chamber The powerplant chamber will be excavated in bedrock about 1,500 m west of the East Pit Reservoir intake structures at an elevation of about 200 m-msl to insure adequate net positive suction head to avoid cavitation when in the pumping mode. The chamber, which will be 21 m wide and 53 m high, will be divided into four compartments each about 122 m long: two for Unit 1 and two for Unit 2. Each compartment will house three Francis-type reversible pump/turbine generation units, each rated at 350-MW generating capacity. Having the powerplant compartmentalized will increase safety and allow the Project to be developed in 1,000 MW increments.

Transformer Chamber The transformer chamber will house twelve three-phase transformers, one for each of the pump/turbine generating units. Because the units will be set deep below the ground surface, the transformers will be located in underground chambers next to the powerplant chambers. Isolating the transformers from the other equipment will provide safety in case of transformer fire. With an excavated opening of about 18 m wide and 18 m high, enough room will exist to allow a transformer to pass along the inplace transformers.

Gate Chamber Tailrace gates will be provided relatively close to the powerplant chamber to permit isolation and dewatering of the pump/turbine generating units. This chamber will be 14.6 m wide, 488 m long, and 21 m high.

Access Tunnel Primary access into the underground chambers will be through a 8.5 m horseshoe shaped access tunnel. Beginning at the west side of the town of Eagle Mountain, the tunnel will be excavated 4,500 m through rock to the powerplant chamber. Access to the powerplant will also be provided by vertical air and electrical cable shafts to the surface above the powerplant.

Waterways Waterways will be excavated out of bedrock. Concrete lining will allow water velocities of up to 6 meters-per-second. For Unit 1, two 8.8 m diameter headrace tunnels, which will include a vertical shaft, will connect the powerplant to the upper Central Deposit Reservoir, and two 7.3 m diameter tailrace tunnels will connect to the lower East Pit Reservoir. For Unit 2, two 11 m diameter headrace tunnels, with vertical shafts, will connect the powerplant to the Black Eagle upper reservoir, and two high pressure tailrace tunnels of 8.2 m diameter will connect to the East Pit lower reservoir. The waterways will be connected to inlet/outlet structures at each end which will be equipped with stoplog gates so that the respective waterway can be dewatered for maintenance.

Mechanical Equipment

Twelve Francis-type pump/turbines, each with a rotational speed of 257 revolutions per minute, will be installed in the powerplant chamber to produce a total output (generation) of 4,000 MW. When the Project is actually constructed, it may be possible to use units which have variable rotational speeds, one for pumping and a lower speed for generating, so as to improve overall efficiency. Inside the powerplant for Unit 1, two 8.8 m diameter penstocks will supply water from the Central Deposit Reservoir though trifurcations to the respective 350 MW units for power generation. The water from each group of three pump turbine units will be combined in trifurcations to flow into two tailrace tunnels. For Unit 2, the two 11 m diameter tunnels would be split in trifurcations to supply the respective six 350 MW units for power generation with the flow combined in trifurcations into two tailrace tunnels.

Electrical Equipment

The electrical system will have twelve pump/generator motors, each rated at 350 MW for generating and 420 MW for pumping. Power from each compartment will be brought to the surface through vertical air shafts by means of solid dielectric 230 kV cables which will lead to busbars in the electrical switchyard.

Switchyard

The switchyard will be located on a level parcel of ground directly above the underground powerplant. The

power output voltage will be transformed up to 500 kV and routed to the power grid through the two 500 kV transmission lines.

Power Transmission Facilities

Two 500-kilovolt (kV) transmission lines will be constructed. One line will be routed toward the south to the vicinity of the SCE Red Cloud Mine Road series condenser station on the SCE Palo Verde to Devers 500 kV intertie, then follow the existing 500 kV line to deliver the power to the SCE Devers substation which is north of Palm Springs, California. Another line may be constructed paralleling the MWD 230 kV transmission corridor to the north, delivering power to the LADWP McCullough Substation south of Henderson, Nevada. A total of about 230 km of transmission lines are planned.

Recreational Facilities

The Project will include recreational facilities for camping, picnicking, swimming, boat docking and launching, fishing and hunting, plus provision for sanitation and waste disposal in the development of the Black Eagle Upper Reservoir.

Table 1. Physical Characteristics of the Eagle Mountain Pumped Storage Project

Central Deposit Upper Reservoir	
Surface area at maximum water elevation	3.30 km^2
Maximum operating water surface elevation	821 m-msl
Minimum operating water surface elevation	714 m-msl
Dead Storage elevation	695 m-msl
Stored Volume	88.8 10^6 m^3
Capacity for power generation	87.0 10^6 m^3
Dead Storage	1.85 10^6 m^3
Black Eagle Upper Reservoir	
Surface area at maximum water elevation	4.86 km^2
Maximum operating water surface elevation	679 m-msl
Minimum operating water surface elevation	550 m-msl
Dead Storage elevation	518 m-msl
Stored Volume	148.0 10^6 m^3
Capacity for power generation	145.6 10^6 m^3
Dead Storage	2.47 10^6 m^3
Lower Reservoir	
Surface area at maximum water elevation	3.44 km^2
Maximum operating water surface elevation	475 m-msl
Minimum operating water surface elevation	268 m-msl
Dead Storage elevation	244 m-msl

Stored Volume	247.0 10^6 m^3
Capacity for power generation	244.3 10^6 m^3
Dead Storage	2.5 10^6 m^3
Dam 1 (Central Deposit Reservoir)	
Crest elevation	826.0 m-msl
Foundation elevation	740.7 m-msl
Length along crest	2,286 m
Crest width	9 m
Upstream slope (reservoir side)	1.75:1
Downstream slope Air side)	1.5:1
Dam 2 (Black Eagle Reservoir)	
Crest elevation	672 m-msl
Foundation elevation	363 m-msl
Length along crest	1,061 m
Crest width	9 m
Upstream slope (reservoir)	1.75:1
Downstream slope (air side)	1.5:1
Dam 3 (East Pit Lower Reservoir)	
Crest elevation	480 m-msl
Foundation elevation	338.3 m-msl
Length along crest	3,200 m
Crest width	9 m
Upstream slope (reservoir)	1.75:1
Downstream slope (air side)	1.5:1
Emergency spillway. Short, low section	100 m
Dike 1 (East Pit Lower Reservoir, south)	
Crest elevation	480.0 m-msl
Foundation elevation	411.5 m-msl
Length along crest	1,463 m
Crest width	9 m
Upstream slope (reservoir)	1.75:1
Downstream slope (air side)	1.5:1

Underground Powerplant

Powerplant chamber (meters)	21W x 549L x 53.3H
Transformer chamber (meters)	18W x 488L x 18.3H
Tailrace gate chamber (meters)	14.6W x 488L x 23.5H

Twelve pump/turbine units

Pump data, Unit 1:	
Maximum head (556.0 m)	429 MW
Design head (445 m)	343 MW
Minimum head (214.7 m)	187 MW
Pump data, Unit 2:	
Maximum head (391.4 m)	512 MW
Design head (313 m)	410 MW
Minimum head (43 m)	57 MW

Turbine data, Unit 1:	
Maximum head (548.3 m)	434 MW
Design head (438 m)	347 MW
Minimum head (241.7 m)	192 MW
Turbine data, Unit 2:	
Maximum head (387.1 m)	520 MW
Design head (310 m)	416 MW
Minimum head (47 m)	63 MW

<u>Waterways</u>

Unit 1: (Central Deposit Upper Reservoir)	
Two high pressure headrace tunnels	9.14 m-diam
Two vertical shafts	9.14 m-diam
Two high pressure tailrace tunnels	7.32 m-diam
Unit 2: (Black Eagle Mine Upper Reservoir	
Two high pressure headrace tunnels	11.0 m-diam
Two vertical shafts	11.0 m-diam
Two high pressure tailrace tunnels	8.23 m-diam

Calibration of Hydrologic and Energy Production Models for High-Head Low-Flow Hydroelectric Plants

Jeffrey E. Twitchell, P.E.[1]

Abstract

Have you calibrated your hydrologic and energy production simulation models lately? Will your hydrologic model accurately predict how your planned or operational projects operate during variable daily flow conditions, extremely wet periods and, more importantly, during critical dry years? Could you have negotiated a better instream flow requirement with the regulatory agencies if you had an accurate hydrologic model that utilized more actual stream flow data?

If you were to refinance your existing operational project(s) or to purchase another party's operational project, would you use the same hydrologic and energy production simulation models that were initially developed to finance the project before commercial operation?

The best available hydroelectric models are those developed from comparing actual operational experiences and data to calculated project performances. The author shares his experiences of comparing optimistic energy production simulation results for financing proposed projects to actual operational performances of the same or nearly identical operational projects. The project models studied in detail are primarily for projects located in the western states of California, Oregon, Washington, and Colorado.

Introduction

This paper stresses the importance of: (i) establishing and maintaining stream flow gages at each of your planned and existing diversion sites; (ii) obtaining as much long-term hydrologic data as possible from within or nearby basins; (iii) utilizing actual turbine/generator and plant performance data; (iv) checking the head losses through your penstock systems; (v) properly accounting for your minimum instream flow releases, which at times may require over releasing for proper

[1] **Western Regional Manager, STS HydroPower, Ltd.**
7311 Greenhaven Drive, Sacramento, CA 95831 (916) 395-8811

regulatory compliance; and (vi) making adjustments for unplanned utility outages, minor equipment failures, and operator inefficiencies. The paper also takes a close look at design mistakes commonly made as a result of poor modeling and data collection techniques. Suggestions are also made on how you may improve your position for lowering your current instream flow requirements with actual stream flow data if your instream flow requirements were partially based on inaccurate synthesized flow data.

The characteristics and benefits of the hydrologic and energy simulation models of run-of-the-river high-head, low-flow hydroelectric facilities are seperated into four different categories: (1) regional and site specific hydrology, (2) maintaining instream flows and accounting for unplanned outages, (3) plant performance characteristics, and (4) future project design modifications. The information presented is based on actual operational data and experiences that have been applied to recalibrate models in basins of 25 square miles or less for existing plants and future plants.

The subject hydrologic models discussed pertain to small hydro tributary basins located in the coastal ranges and western Sierra slopes of California, the western slopes of the Cascades in Washington and Oregon states and the western slopes of the Colorado Rockies.

Regional and Site Specific Hydrology

Every possible effort should be made to install a stream recording gage and record flows at the diversion site of any new or future run-of-the-river site. Many times gages are within the same basin, but placed a significant distance from the diversion site, and as a result, may require significant calibration from within the basin itself. If your project basin is influenced by variable snow-runoff conditions in the higher elevations, and/or by spring fed characteristics from anywhere within the basin, a recording station at the diversion will answer a lot of questions. The recording station should record stream levels or flows at least once every hour. (USGS standards call for readings at every 15 minute interval or 96 readings per day.) The stream flow gage should accurately measure flows between (i) the natural minimum stream flows and (ii) the maximum turbine design flow plus the maximum instream flow requirements. The stream gage must be able to track the daily diurnal (daily fluctuation) in the smaller high-head low-flow streams. The diurnal in some high-elevation, snow-fed basins may have significant changes in flow that may distort the average daily flow available for generation. It is also important to track the rising and receding hourly hydrographs of major storm events that occur in the basins.

All too often, project proponents are forced to simulate small drainage basin daily flows from large basin flows. Sometimes the large basin gages are in adjoining neighboring basins and sometimes they are regional basins that can be over 50 to 100 miles away from the proposed site. The larger river basin flows should only be used to determine annual and long-term trends in the basin or regional critical flow periods. They should not be used or scaled down to model daily flows for significantly smaller basins . The daily, weekly, and monthly dynamics

of flows in the high gradient streams are not easily detected nor visible in large basin gages. Average monthly flows always tend to overstate the availability of water, thus daily flows must always be used to model run-of-the-river high-head, low-flow plants.

A daily average flow in a high gradient stream at the beginning or end of a high run-off event may be significantly less than the median flow for the same day and it may easily overstate the amount of water available for generation. A year or two of daily data collected from the point of diversion is much better then 10 or 20 years of data collected from a significantly larger neighboring basin gage. The short-term one or two-year record collected at the diversion can later be easily correlated to a larger basin with a longer period of record. When calibrating or correlating data from neighboring basins or from within the basin, individual correlation factors should be developed for each month of data, and if possible should be based not only on run-off but from precipitation records as well. Do not let the large flow run-off events exceeding the design capacity of your proposed project skew the monthly correlation factors your are developing for your design flows. Significant differences in month-to-month correlation factors (as much as 50%) have also been experienced from data collected at the mouth of a small 10 square mile drainage area to data collected in the same basin at an elevation 1300 feet higher that only measured approximately seven square miles of the same drainage area. The major variability is accounted for by the lower elevation gage showing significantly higher run-off flows when snow fall was occurring in the upper basin while rainfall was running off in the lower basin. The opposite was taking place in the late spring months when nearly all of the drainage run-off was occurring from the upper reaches while the lower reaches of the basin added very little contribution to the season run-off.

Accounting for Instream Flow Requirements and Unplanned Outages

The primary purpose for installing a stream gage is for determining the availability of water for generation planning purposes, but it can also be used to show the natural flow conditions that are experienced in the stream by fish and wildlife. If a stream gage was not installed prior to negotiating your minimum instream flow requirements it is very possible that the true limiting factors to the local fishery were not known. In many cases, the natural conditions are not always ideal for fish, and the data can be used to show that there may be far more critical limiting factors occurring naturally than would occur under hydro operations containing recommended minimum flow requirements and limited ramping rates. In some cases a stream may have ideal fishery conditions for ten months out of the year, but two months of either extremely low flows or high peak flows may be more detrimental to the survival of the sustained fishery. Providing year-round optimum or near optimum habitat flow conditions with instream flows preferred by the resource agencies should be revisited with the agencies if you can demonstrate that natural conditions are still substantially more limiting to the fishery.

When developing an energy simulation model, one always runs sensitivity analyses to determine what impacts various instream flow requirements will have on the net energy production. In doing so, one should always increase the required or anticipated instream flow requirements by as much as 10 percent in the model to

account for manual and automatic operator errors. With strict compliance requirements, operators will tend to over release the minimum requirements. Operation experience also indicates that perfect conditions do not always exist that allow for continuous minimum releases, particularly if the stream is rising and/or decreasing quicker than your operators and plant equipment can respond.

Most high-head, low-flow, run-of-the-river reaction turbine/generator units will operate down below 10% of its full design capacity. However, during days when the stream flow transitions between: (i) the minimum instream flow requirement and the minimum turbine operational flow, or (ii) the collective maximum design flow and the required instream flows, the daily reported average flow may overstate the water availability for hydro generation. These transition days are rarely accounted for in models and can be significant if the stream contains highly variable flow conditions. This small loss of generation should be accounted for in a similar fashion as one estimates plant outages for major high water events.

Depending upon the configuration of each individual project, it is not uncommon for one, two, or five-year storm events in the high gradient streams to trip the plant off line. The storm events will carry either significant amounts of bed load or woody debris that will temporarily clog intake systems or require curtailment of diversion flows. These storm events must be accounted for collectively with power transmission outages that will commonly occur during peak storm run-off events as well. The models can easily program an upper flow cut-off limit for high water events, but one must use his best judgment on developing a monthly energy deduction of 0% to 10% that is commensurate with the history of unplanned utility line outages occurring within the immediate project area.

Proper consideration must also be given to lost generation occurring immediately after unplanned outages due to ever increasing compliance requirements of instream ramping rates.

The models must also deduct for minor equipment failures and minor operator inefficiency errors that are more likely to occur during high run-off events.

Plant Performance Characteristics

Actual on-line operational data should be used as much as possible to chart plant performance and to predict future generation from the existing plant and future projects. Look closely at the actual long-term instantaneous Kwh production statements versus instantaneous Kw outputs occurring only under ideal conditions.

Closely review the turbine(s) availability of net head over the full range of operating flows to develop accurate head loss coefficients and to calibrate your system's head losses for future models. Check your head losses over time to see if they are staying constant or are increasing proportionately with service life of conduit systems.

Use actual net Kw produced from the plant for the full range of turbine generator flows versus published and/or guaranteed efficiency curves. Again, it is

important to maintain accurate flow measuring devices to determine precise flow amounts being routed through the plant and within the bypass stream reach. With high-head, low-flow plants, one should be able to develop a simple model equation or chart for your plant based on actual net Kw produced versus flow, or average daily Kwh versus average daily flow.

Future Plant Design and Modelling Modifications

When comparing actual generation performances of existing plants to pre-project models one can easily detect common modelling errors. In addition to identifying modelling errors one can detect operational errors and common plant design errors that can be eliminated in the future. The best check for detecting errors is to run the actual stream gage flow data collected during project operations through the model and closely compare the model results with the ongoing plant performance.

Plant modifications or design improvements for future projects may consist of expanding or adjusting the operational limits of the turbine generator unit(s). If the power being generated is more valuable during the low flow months one may want to consider lowering the lower operational limit or installing a small, separate turbine generator unit to capture valuable low flows. This could prove to be very valuable during extremely dry years.

Conclusions

Do not over estimate the energy production capabilities of new small run-of-the-river hydroelectric sites in the western states by using large basin hydrologic data or average monthly flows. Install a stream gage at the point of diversion and take the guess work out of your hydrologic model. One or two years of daily flow records at the point of diversion is more valuable than ten years of data from an adjoining larger basin. Review the operational records of other similar projects before relying solely on theoretical computed values.

Continuously update your energy simulation models by comparing calculated energy values with actual net energy deliveries from operational plants containing accurate flow records.

In Search of Navigable Waters

Dr. Henry G. Ecton[1]

Abstract

Section 23(b)(1) of the Federal Power Act requires that waterpower projects be licensed if they are located on navigable waters of the United States. "Navigable waters," as defined under Section 3(8) of the Act, means those parts of streams or other bodies of water over which Congress has jurisdiction under its authority to regulate commerce with foreign countries and between states. A waterway is navigable if it (or part of it) is or was used (or is suitable for use) for the transportation of persons or property in interstate or foreign commerce, even if the transit is interrupted by falls, shallows, or rapids requiring portage. If the waterway has been improved by Congressional authorization, or has been recommended for improvement to Congress, this also fulfills the requirement.

This paper presents a historical overview of the interpretations of the term. It reviews court and Commission decisions, from Daniel Ball to David Zinkie. It examines "historic navigation" and explains its role in current review procedures. The paper examines the navigation studies of two waterways, one navigable and one not navigable, to explain why such judgments were made. It also reviews how the Commission's concept of navigable waters differs from other agencies, state and Federal, and why. Finally, the paper presents the current Commission understanding of navigable waters.

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[1] Historian, Project Evaluation Branch, Office of Hydropower Licensing, Federal Energy Regulatory Commission, 825 N. Capitol St., NE, Washington, D.C. 20426.

Introduction

A Federal Power Commission lawyer, caught in a downpour on Washington's best-known thoroughfare one summer day in 1964, saw a stick of wood float by in the gutter. "By gosh," he exclaimed, "we can just about take jurisdiction of Pennsylvania Avenue."[2]

This gentle humor was aimed at the broad view taken by the Federal courts of "navigable waters of the United States."

The term "navigable waters" has generated considerable, and often heated, discussion, since Section 23(b) of the Federal Power Act (FPA) requires that waterpower projects be licensed if they are located on navigable waters of the United States. Section 3(8) of the FPA defines "navigable waters" as those parts of streams or other bodies of water over which Congress has jurisdiction under its authority to regulate commerce with foreign countries and between states, and which are used or suitable for use to transport people or property in interstate or foreign commerce, even if the transit is interrupted by falls, shallows, or rapids compelling land carriage. Also, if the waterway has been improved, or recommended for improvement by Congress, this also fulfills the requirement.[3]

Although the definition of "navigable waters" appears straightforward, the Federal courts have repeatedly been asked to clarify and interpret this term.

Legislative Development

Before the enactment of the Federal Water Power Act in 1920, the construction and operation of dams in navigable waters (and in non-navigable tributaries whose flows affected such waters) were regulated under four general statutes: Section 7 of the River and Harbor Act of 1890, as amended;[4] sections 9 and 10 of the River and Harbor Act of

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[2] Wall Street Journal, October 27, 1964.

[3] Gibbons v. Ogden, 22 U.S. 1 (9 Wheat. 189)(1824). Also 16 U.S.C. § 796(8) (1982).

[4] Act of September 19, 1890, ch. 907, sec. 7, 26 Stat. 454, amended by act of July 13, 1892, ch. 158, sec. 3, 27 Stat. 110. See also U.S. v. Rio Grande Irrigation Co., 174 U.S. 690 (1899)

March 3, 1899;[5] the General Dam Act of 1906;[6] and the General Dam Act of 1910 (which amended and re-enacted the General Dam Act of 1906.)[7] The first of these statutes made it unlawful, without permission of the Secretary of War, to build any structures of the types named therein, in specified areas within the navigable waters of the United States, that would obstruct navigation, commerce, or anchorage. The 1899 Act, which superseded the 1890 Act, made the lawfulness of structures in navigable waters dependent upon the consent of the Congress and the approval of plans by the Chief of Engineers and the Secretary of War. The 1906 Act prescribed in considerable detail the conditions which were to govern the construction, maintenance, and operation of dams and associated facilities in navigable waters. The 1910 Act added new requirements to the provisions of the General Dam Act of 1906. Among these was a requirement that in acting on plans, the Chief of Engineers and the Secretary of War should consider the effect of the proposed structure on a comprehensive plan for improvement of the waterway involved.

Although the Federal Water Power Act of 1920 expanded the Federal authority delegated under these previous acts, the power of the new Federal Power Commission (FPC), consisting of the secretaries of War, Interior and Agriculture, remained limited. The Commission was not authorized to employ any personnel other than an Executive Secretary.[8] Members of the Staffs of the three executive departments were assigned to discharge the Commission's responsibilities. Each year the Commission sought Congressional authorization to employ its own staff. This request was finally granted in 1928, when 29 employees were transferred from the three executive departments to the Commission. However, the constant increase in workload made it obvious that the three Secretaries could not devote the necessary time to Commission duties.[9] In 1929, President Herbert Hoover recommended creation of an independent agency. This was accomplished by statute adopted on June 23, 1930.[10]

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[5] Act of March 3, 1899, ch. 425, sections 9 and 10, 30 Stat. 110.

[6] Act of June 21, 1906, ch. 508, 34 Stat. 386-387.

[7] Act of June 23, 1910, ch. 360, 36 Stat. 593-596.

[8] 41 Stat. 1063.

[9] It was estimated that the Secretaries devoted only about 5 hours per year to their FPC duties. 72 Congressional Record 8752 (1)

[10] 46 Stat. 797.

The Public Utility Holding Company Act of 1935[11] incorporated the Federal Water Power Act into Part I of the new Federal Power Act, and added Parts II and III. A number of substantive changes were made in Part I at this time in order to clarify and broaden the Commission's authority over water power projects.

The Federal Water Power Act of 1920 made it unlawful to operate a hydroelectric project in navigable waters without a license from the FPC. A comparable prohibition as to dams was contained in the River and Harbor Act of 1899. However, while the chief emphasis of the earlier acts had been on the protection of navigation, the avowed purpose of the Federal Water Power Act was to bring the hydroelectric power industry within the ambit of Federal regulation, with navigability being one touchstone of jurisdiction, but otherwise of secondary concern.

Federal Courts and "Navigable Waters"

Congressional authority to regulate navigable waters has been expanded by a series of court decisions. One of the first, and now classic, decisions is The Daniel Ball (1870), in which the Supreme Court adopted the "navigation in fact" test; that a river is navigable if it is used or is susceptible of being used, in its ordinary condition, as an actual avenue of commerce.[12] In a second case, The Montello (1874), the Supreme Court declared that "the true test of the navigability of a stream does not depend on the mode by which commerce is, or may be, conducted nor the difficulties attending navigation." Rather, said the Court, "[t]he capability of use by the public for purposes of transportation and commerce affords the true criterion of the navigability of a river, rather than the extent and manner of that use."[13]

Later, in Economy Light & Power Co. v. United States (1921), the Supreme Court made clear that "[n]avigability, in the sense of the law, is not destroyed because the watercourse is interrupted by occasional natural obstructions or portages; nor need the navigation be open at all seasons of the year, or at all stages of the water." Further, the Court stated that obstructions, even those which are artificial, as well as

[11] This act amended the Federal Water Power Act of 1920, by 49 Stat. 838 (1935) U.S.C. Supp. V. Title 16 § 791a et seq., by which it became known as the Federal Power Act.

[12] The Daniel Ball, 10 Wall. 557 (1870).

[13] The Montello, 87 U.S. (20 Wall.) 430 (1874).

changes in transportation mode which result in a river's disuse for many years, do not destroy its status as a navigable waterway.[14]

The Federal Power Commission

Despite these court rulings, and because of the lack of staff, between 1920 and 1930 the Commission disclaimed jurisdiction in a majority of the cases in which declarations of intention were voluntarily submitted to it. During that period, the criterion employed by the Commission was apparently the same as that employed by the Army Corps of Engineers under the river and harbor acts--namely, whether or not any actual or probable navigation was genuinely endangered by the proposed hydroelectric project. Thus, it appears that the question of whether the public interest required Federal supervision over a project was the most important factor in determining jurisdiction rather than legal navigability or the effect thereon. In 22 of the cases where jurisdiction was declined, the rivers had been used for the transportation of forest products, but were found not to be "navigable waters" as defined in the Federal Water Power Act at the sites of the projects involved. Included among the rivers were the Androscoggin, Saco, Connecticut, Menominee, Chippewa, and Flambeau, all of which have been found to be navigable waters of the United States subsequent to 1942.

On December 23, 1937, the Commission instituted a general investigation (IT-5501) to determine what water power projects were being operated and maintained with valid Federal authority, on waters over which Congress has jurisdiction. The IT-5501 studies were comprehensive, with the researchers often spending months on location researching the river basins. These reports have served as the basic foundation for most of the Commission's navigation studies.

Despite this undertaking, between 1931 and 1942 the Commission took jurisdiction in only one case based on navigable waters--Cedar River, Iowa--and in 13 cases based on effect on interstate and foreign commerce, without findings as to navigable waters. The Commission declined jurisdiction in 25 cases on the basis of no effect on interstate or foreign commerce. In six of the 25 cases, the streams were declared not navigable waters. Four other cases were disposed of without a finding.

Court cases continued to expand the definition of navigable waters. On December 16, 1940, the Supreme Court gave the term "navigable waters" in the Federal Power Act a broader construction than that laid

[14] Economy Light & Power Co. v. U.S., 256 U.S. 113, 122 (1921).

down in The Daniel Ball and progeny. In the landmark Appalachian[15] case the Court established that a finding of navigability need not depend on a river's ordinary condition. The Court also made clear that navigability can be found "despite the obstruction of falls, rapids, sand bars, carries or shifting currents." It stated that "absence of use over long periods of years, because of changed conditions, . . . does not affect the navigability of rivers in a constitutional sense." Finally, the Court declared that lack of commercial traffic is not "a bar to a conclusion of navigability where personal or private use by boats demonstrates the availability of the stream for the simpler types of commercial navigation."

Prior to 1943, the Commission had declined jurisdiction in 22 cases despite evidence that the stream involved had been used for transporting forest products in interstate commerce, and had relied exclusively on logging in only 3 of 20 cases in which it had found streams to be navigable. One of these 3 cases, the Tomahawk decision, was appealed to the United States Circuit Court of Appeals following a Commission decision of navigability in 1943. The case, concerning logging on part of the Wisconsin River, was upheld by the Court of Appeals in 1945,[16] which stated that "it is well settled that the floating of logs, in the course of a continuous movement from one State to another, is interstate commerce, and may be a sufficient use for the purposes of commerce," and established that seasonal logging is sufficient by itself to support a finding of navigable waters. Since this decision, the Commission has consistently used evidence of the transportation of forest products in interstate or foreign commerce, to find that streams are "navigable waters" of the United States.

During the Second World War, there was little Commission activity in this area. However, in 1946, the Commission renewed its investigation of navigable waters. This renewed activity attracted a political response. In 1947, the introduction of HR 2973, 80th Congress (one of the so-called Miller Bills) attempted to curtail this new activity. The measures attempted to narrow the Commission's jurisdiction by amending Section 3(8) (definition of navigability) and Section 23(b). In considering the effect of HR 2973, it was estimated that 127 of 155 projects under major license, all 33 minor part projects, all 162 minor projects, and all 312 transmission line projects then outstanding would probably be removed from Commission jurisdiction.

[15] United States v. Appalachian Electric Power Co., 311 U.S. 377 (1940).

[16] Wisconsin Public Service Corporation v. Federal Power Commission, 147 F.2d 743 (7th Cir.), cert. denied, 325 U.S. 880 (1945).

Although the Bill was supported by the electric industry and some States, it did not pass. However, largely as a result of the Miller Bill, time spent on the unlicensed project program after 1949 became very small and generally was confined to cases started in prior years and on streams in which additional construction had been proposed.

The Commission's investigation of navigable waterways received new life in the 1960s. In 1965, the Supreme Court handed down the Taum Sauk decision, which expanded the Commission's licensing responsibilities to projects located on non-navigable streams and generating energy for an interstate power system.[17] As a result, the Commission increased its review of unlicensed projects, seeking those that might be operating without a valid Federal permit. As part of that effort, the Commission continued its review of waterways, seeking projects that might be located on "navigable waters."

The Federal Energy Regulatory Commission

In 1977, following the creation of the Federal Energy Regulatory Commission[18] (FERC) as the successor agency to the FPC, questions about navigable waters became of secondary importance. The Chairman of the Commission decided that, because of limited personnel resources and the expanding hydro licensing workload, investigations into navigable waters must be of secondary concern. However, in 1986, under the direction of a succeeding Chairman, the Office of Hydropower Licensing began investigating unlicensed projects to determine whether they were located on navigable waters and if they fell under the Commission's licensing jurisdiction, according to Section 23(b).

Research Guidelines

Since 1986, the Commission has conducted reviews of more than 94 waterways. When reviewing a waterway the FERC historian takes the long view of navigability, since waterways can change dramatically over centuries. When seeking evidence of the navigability of a waterway, oral traditions as well as later written social and economic histories can

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[17] Union Electric Company v. FPC, 381 U.S. 90 (1965).

[18] The Federal Energy Regulatory Commission (FERC) was established by the Department of Energy Organization Act, which was enacted on August 4, 1977. Public Law 95-91 Stat. 565. The effective date of the provisions of the Act was established October 1, 1977, by Executive Order No. 12009, signed by the President on September 13, 1977 (42 F.R. 46267, September 15, 1977).

provide information concerning the transport of persons and property in interstate commerce. The research includes a review of primary and secondary sources, including published histories, treaties, newspapers, periodicals, historical journals, lumber journals, trade reports, historic maps, census data, court cases, unpublished monographs and dissertation, and other relevant documents.

Using Court and Commission guidelines on navigability, the researcher establishes the date of settlement of towns and counties on the waterway and examines the establishment of industries, such as textiles, lumber, and mining. The researcher also examines how agricultural commodities, such as cotton and tobacco, were moved to market. To the researcher, the absence of evidence of waterway navigability is as important as the presence of such evidence. Thus, information must be included on alternative routes used to move goods and people to and from place to place. Some evidence, such as State legislation designed to promote navigation on a waterway, is not conclusive evidence. During the eighteenth and nineteenth centuries States frequently passed such legislation with the hope that a navigable waterway might be developed, but the waterway remained navigable only on paper. Other agencies and departments of the State and Federal governments also provide good information on the use of waterways. The evidence gathered by the U.S. Army, Corps of Engineers, for example, concerning the navigability of a waterway, is useful in assisting the Commission in making its determination. However, the decisions by other agencies are not binding on the Commission.[19]

Two Case Studies

The Court's definition of a navigable waterway has changed little during the past fifty years. One notable difference is that the Commission now considers recreational boating as a criterion, assuming the boating is a commercial venture and affects interstate commerce. A recent decision, the David Zinkie case[20], determined that the Fawn River, a small river in Indiana, was a navigable waterway, based on use of the river by recreational boaters.[21] The river, which meanders through Michigan and

[19] Pennsylvania Water & Power Co. v. FPC, 123 F.2d 155 (1941), cert. denied, 315 U.S. 806 (1942).

[20] See 46 FERC ¶ 62,027 (1989) and 53 FERC ¶ 61,029 (1990).

[21] This course of action had been suggested several decades earlier. In United States v. Appalachian Electric Power Co., 311 U.S. 377, 416 (1940), the Court stated: "Nor is lack of commercial traffic a bar to a

Indiana, has been used for recreational boating since 1916. It was clear from the navigation report that the recreational use of the river takes place on a commercial, interstate scale. According to the report:[22]

> It can be used by canoeists for short or long distances, or by fishermen who may also travel along the river. Because the river crosses back and forth across the state line so frequently, canoeists moving along the river also do so. The requirement for fishing licenses from Indiana and Michigan both, for fishermen fishing the length of the river, clearly documents the interstate nature of the activity.

The presence of recreational boating or logging is not ipso facto confirmation of the navigability of a stream, however. In its review of the Harrisville Project, located on the Montello River, the Commission found evidence of recreational boating and logging, but no linkage to interstate commerce.[23]

Conclusion

During the past 125 years, the definition of navigable waters has been discussed and debated numerous times. Court decisions have set forth a general view of what constitutes navigable waters: It has been determined that (1) a river is navigable if it was used or was susceptible to use in its ordinary condition as an actual avenue of commerce;[24] (2) navigability is not destroyed by obstructions or disuse for many years;[25] (3) the lack of commercial traffic is not a bar to the conclusion of navigability where personal or private use demonstrates the availability of

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conclusion of navigability where personal or private use of boats demonstrates the availability of the stream for the simpler types of commercial navigation." See also City of Centralia, Wash. v. FERC, 851 F.2d 278, 282 (9th Cir. 1988); Puget Sound Power & Light Co. v. FERC, 644 F.2d 785, 788 (9th Cir. 1981).

[22] Navigability of Fawn River (Star Mills Hydropower Project) LaGrange County, Indiana, Federal Energy Regulatory Commission, December 1988.

[23] 50 FERC ¶ 62,114.

[24] The Daniel Ball, Supra.

[25] Economy Light & Power Co., Supra.

the river for simpler forms of commercial navigation;[26] and (4) the seasonal floatation of logs in interstate commerce is sufficient to determine that a river is navigable.[27]

"Navigable waters" is not a static term, as demonstrated in this paper. The definition has been fluid and changing. However, certain basic criteria remain unchanged: A waterway is navigable if it is presently being used or is suitable for use, or it has been used or was suitable for use in the past, or it could be made suitable for use in the future by reasonable improvements. And, despite the humorous view expressed by the Federal Power attorney in the first paragraph, floatability is still not the same as navigability.[28]

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[26] Appalachian Electric Power Co., Supra.

[27] Wisconsin Public Service Corp., Supra.

[28] The material in this paper is from historical overviews and collections put together by employees of the Federal Power Commission and the Federal Energy Regulatory Commission, including John C. Mason, W. R. Farley, Frank Goodman, Steven Mathews, Clinton Hull, Joseph B. Hobbs, Etta Foster, Linda Gilbert, Kristina Nygaard, and John Clements. Any errors, however, are mine alone.

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WATERPOWER '93

W. David Hall, Editor

Proceedings of the International Conference on Hydropower, Nashville, TN, August 10- 13, 1993

The perceived simplicity of production of hydropower gives it a less dramatic public image than that of some other sources of energy. While hydropower is the most efficient energy technology, opportunities exist to improve its performance and its existence with the environment. This three-volume set surveys up-to-the-minute information and contemporary issues concerning hydroelectric power. Contributors discuss topics ranging from economics and finance to licensing and legal concerns, from dam safety to computer applications. Papers address water quality, diversion screens, and instream flows; turbine impacts on fish and fish protection; the design, manufacturing, and testing of turbines; and the rehabilitation and modernization of various hydroelectric plants, among many other subjects.

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